1

Author: John R. Lohr, Sr.

Title: Evaluation of "Formwall"--
A Post-Tensioned,
Dry-Stacked Masonry System

Rank: Capt.

Branch: USAF

Date: 1992

Pages: 89

Degree: Master of Science

Institute: The Pennsylvania State University

ORT DOCUMENTATION PAGE	Form Approved OMB No. 0704-0188

Public reporting burden for this collection of information is estimated to average 1 hour per response, including the time for reviewing instructions, searching existing data sources, gathering and maintaining the data needed, and completing and reviewing the collection of information. Send comments regarding this burden estimate or any other aspect of this collection of information, including suggestions for reducing this burden, to Washington Headquarters Services, Directorate for Information Operations and Reports, 1215 Jefferson Davis Highway, Suite 1204, Arlington, VA 22202-4302, and to the Office of Management and Budget, Paperwork Reduction Project (0704-0188), Washington, DC 20503.

1. AGENCY USE ONLY (*Leave blank*)	2. REPORT DATE 1991	3. REPORT TYPE AND DATES COVERED THESIS/XXXXXXXXXXXX

4. TITLE AND SUBTITLE
Evaluation of "Formwall"-- A Post-Tensioned, Dry-Stacked Masonry System

5. FUNDING NUMBERS

6. AUTHOR(S)
John R.Lohr, Sr., Captain

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
AFIT Student Attending: Pennsylvania State University

8. PERFORMING ORGANIZATION REPORT NUMBER
AFIT/CI/CIA-91-128

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)
AFIT/CI
Wright-Patterson AFB OH 45433-6583

10. SPONSORING/MONITORING AGENCY REPORT NUMBER

11. SUPPLEMENTARY NOTES

12a. DISTRIBUTION/AVAILABILITY STATEMENT
Approved for Public Release IAW 190-1
Distributed Unlimited
ERNEST A. HAYGOOD, Captain, USAF
Executive Officer

12b. DISTRIBUTION CODE

13. ABSTRACT (*Maximum 200 words*)

14. SUBJECT TERMS	15. NUMBER OF PAGES 89
	16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT

The Pennsylvania State University

The Graduate School

Department of Architectural Engineering

EVALUATION OF "FORMWALL"--
A POST-TENSIONED,
DRY-STACKED MASONRY SYSTEM

A Thesis in

Architectural Engineering

by

John R. Lohr, Sr.

Submitted in Partial Fulfillment
of the Requirements
for the Degree of

Master of Science

May 1992

I grant The Pennsylvania State University the nonexclusive right to use this work for the University's own purposes and to make single copies of the work available to the public on a not-for-profit basis if copies are not otherwise available.

J. R. Lohr, Sr.

John R. Lohr, Sr.

We approve the thesis of John R. Lohr, Sr.

Date of Signature

M. Kevin Parfitt — Nov. 27, 1991
M. Kevin Parfitt
Assistant Professor of
Architectural Engineering
Thesis Advisor

Louis F. Geschwindner — 11/27/91
Louis F. Geschwindner, Jr.
Associate Professor of
Architectural Engineering

Harry H. West — Nov. 27, 1991
Harry H. West
Professor of Civil Engineering

Paul A. Seaburg — Nov 27, 1991
Paul A. Seaburg
Professor of Architectural Engineering
Head of the Department of
Architectural Engineering

ABSTRACT

This study examined the constructibility, feasibility, and flexural behavior of a new and untested post-tensioned, dry-stacked concrete masonry wall system--know as "Formwall"--as proposed by the National Concrete Masonry Association. The evaluation was conducted through mathematical predictions, wall construction, and experimental testing. To substantiate the findings, comparisons were made between the dry-stacked "Formwall" system, a conventional nonreinforced (plain masonry) wall panel with Type "S" mortar, and a dry-stacked wall panel using conventional six-inch concrete masonry units. Based on the test results of this study, the proposed "Formwall" system has virtually no structural capacity due primarily to stability issues stemming from problems with the geometric shape of the face shells and lack of composite action between the face shells and ties. Notwithstanding, it is the author's opinion that there is merit in pursuing the development of a dry-stacked masonry system. And when perfected, post-tensioned dry-stacked masonry could result in significant savings for roadside barriers, landscaping elements, basement walls for low income housing, and temporary structures.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

ACKNOWLEDGEMENTS

I would like to express my profound appreciation to Professor Kevin Parfitt for his support and guidance throughout the research and to the thesis committee as a whole for their guidance when problems and difficulties arose.

I am indebted to Mr. Jorge Pardo, Director, Product Development Division of the National Concrete Masonry Association, who conceived and developed the dry-stacked, "Formwall" masonry system, coordinated the donation of materials, and provided funding. In addition to Mr. Pardo, I would like to express my gratitude to Mr. James P. McCallion, President, RJD Industries, Inc., and Mr. Art DeVecchis, Founder of E. DeVecchis and Sons, both of whom donated materials and equipment in support of the research.

A special thanks goes to Mr. Jim Smith for taking time out of his busy schedule to construct the nonreinforced wall panel and to Mr. Stanley B. Wall and Melissa A. Marsico for their assistance in constructing and testing the wall panels.

Finally, I want to thank the Air Force Institute of Technology for providing funding and support which was critical to my participation in the Architectural Engineering graduate program at The Pennsylvania State University.

Chapter 1

INTRODUCTION

Although masonry is one of mankind's oldest and most popular durable structural building materials, it has only been in the last few decades that scientific principles have been successfully applied to masonry design (Lenczner 1972, ix; Orton 1986, vii). For centuries, structural design of masonry walls was based on arbitrary limits of thickness related to unsupported height and horizontal span as established by control authorities such as building codes (NCMA-TEK Bulletin 27 1971). Although height to thickness (h/t) ratios proved to be adequate, they were not intended to "insure structural performance to elements subjected to high lateral forces due to wind, earthquake, and soil pressure" (NCMA-TEK Bulletin 113A 1977). Recognizing the limitations of empirical design and the validity of engineering practice, the 1985 Uniform Building Code (UBC) dramatically brought an abrupt halt to controlling wall height only through an empirical h/t limit and codified, for the first time, what is commonly referred to as the "rational" approach to engineered masonry design. In other words, ". . . the designer merely needs to show that the structural integrity of the wall can be maintained when subjected to the various combinations of dead, live, and lateral loads" (Schneider and Dickey 1987, 212).

Engineered masonry design is used for all types of structures subjected to a wide variety of forces and loading conditions. Based on allowable stresses, combined loading, and empirical rules, engineered

masonry design is premised on the principles of working stress design and engineering mechanics.

Like concrete, masonry is strong in compression, weak in tension, and designed under the assumption that the masonry develops little to no tensile stress. "Therefore, nonload-bearing walls or walls with low normal forces exhibit a poor cracking behavior and a low ultimate strength" (Ganz 1989, 165). To overcome these disadvantages, masonry can be reinforced or post-tensioned. Post-tensioning, which is an essential process for this study, offers the possibility to actively introduce a desired level of axial load in a wall to enhance strength, performance, and durability of masonry structures. Restated, post-tensioning widens the application of masonry and encourages experimental use in dry-stack masonry.

1.1 Post-Tensioned Masonry

The concept of post-tensioning masonry is not new to the building industry. In fact, it dates back to the 1820s when Brunel employed post-tensioned brickwork in the construction of air shafts for the Thames River Tunnel (Ostag 1986, 2). "The project involved the construction of vertical tube caissons of 15m diameter and 21m height. The 0.75m thick brick walls were reinforced and post-tensioned with 25mm diameter wrought iron rods" (VSL International LTD, 4). Since the Thames River Tunnel project, there has been quite a bit of research and a number of applications of prestressed masonry conducted both in and outside the United States; however, very little, if any, utilize post-tensioned dry-stack masonry. Consequently, the author is unaware of any information concerning post-

tensioning or prestressing dry-stacked masonry other than what is available through the National Concrete Masonry Association which has been experimenting with dry-stack masonry for the last few years.

1.2 Purpose and Scope

This study is concerned with mildly post-tensioned, dry-stacked concrete masonry. More specifically, this study evaluates the overall performance of a new and untested post-tensioned, dry-stacked concrete masonry wall system known as "Formwall" as proposed by the Innovative Design Research Division of the National Concrete Masonry Association (NCMA), Herndon, Virginia.

The purpose of the study is to examine the constructibility, feasibility, and flexural behavior of the "Formwall" system through a variety of experimental methods. To aid in formulating conclusions, comparisons are made between conventional concrete masonry construction, the dry-stacking of conventional concrete masonry units, and the dry-stacked "Formwall" system.

Because "Formwall" is a new and untested masonry system, coupled with the fact that very little, if any, published information is available for post-tensioned, dry-stack masonry, a logical progression of events was planned. First, conventional design/analysis methodologies for both concrete masonry and prestressed concrete were examined to determine relevancy, adaptation, and use. Second, the physical requirements--characteristics and geometric properties--of the "Formwall" concrete masonry units were examined to aid in developing design/analysis methodologies and establish minimum requirements.

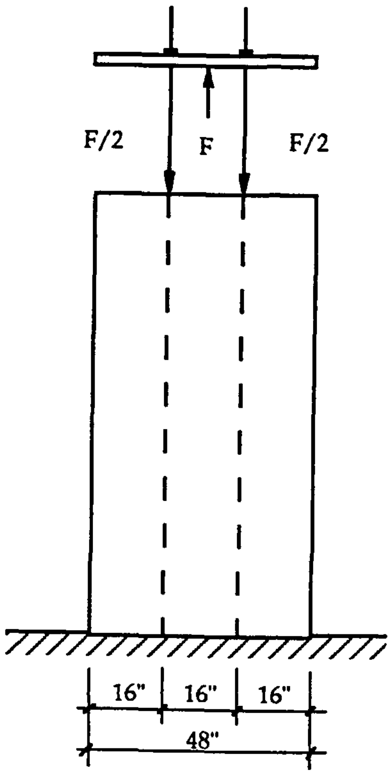
These properties include measurements and dimensions, compressive strength, section modulus, and effective area. Finally, wall panels were constructed and loaded to examine the constructibility and feasibility of a dry-stacked, groutless masonry system.

In addition to evaluating the "Formwall" system, an evaluation of utilizing nonmetallic tendons was conducted. It is not the intent of this study, however, to analyze or collect data concerning the tendons, but rather to investigate the feasibility of utilizing them. For obvious reasons, nonmetallic tendons are an ideal material for reinforcing a mortarless, groutless masonry system. For purposes of this research, however, the wall panels were constructed and loaded using steel tendons. Reasons for not using nonmetallic tendons are discussed in Section 5.8, Evaluation of *Nonmetallic Tendon Anchorage System*.

All tests conducted in this study are in accordance with prescribed methods (but modified where necessary) specified by the American Society for Testing and Materials (ASTM) and principles of engineering mechanics.

1.3 Limitations

There are several limitations to this study, but none as important as the "Formwall" concrete masonry system buckling during the post-tensioning phase. The "Formwall" masonry system, as proposed by NCMA, failed to support a post-tensioning force of 200 pounds (100 pounds per tendon). Figure 1.1 depicts the application of the post-tensioning force. According to preliminary calculations (refer to

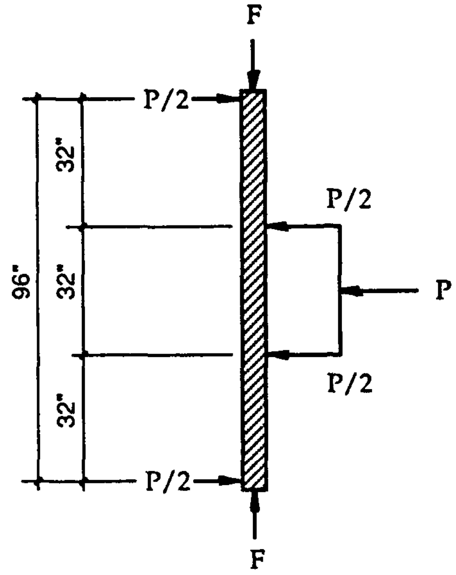


F = Axial load applied to introduce post-tensioning force to tendons.

Figure 1.1 Concentrated Axial Load Used to Introduce Post-Tensioning Force.

Section A.2 of Appendix A), the post-tensioning force required to resist the equivalent applied lateral load of 20 pounds per square foot, which was applied as two equal concentrated loads symmetrically placed as depicted in Figure 1.2, is dependent upon the relationship between the ties and face shells--composite or noncomposite action. A conservative post-tensioning force of 5,400 pounds (2,700 pounds per tendon) was targeted for this study based on the preliminary calculations and a margin of safety. In other words, not fully understanding the behavior or capacity of the "Formwall" wall panel, the post-tensioning force was applied with extreme care. As a result of the wall buckling during the post-tensioning of the wall panel, the author was unable to study the flexural capacity of the "Formwall" system, develop an understanding of the relationship between the ties and face shells or of how the ties might influence the transfer of stresses, derive or validate physical characteristics and geometric properties, or study the behavior of creep and prestress loss. This somewhat unexpected failure of the wall undermined the study in terms of collecting quantitative data. Notwithstanding, valuable qualitative data is presented to aid future research in the field of dry-stacked masonry.

Another limitation, which must be overcome before this system can possibly be realized and placed in full commercial use, is the absence of an "off the shelf" device to "lock off" or sustain a post-tensioning force in nonmetallic tendons. The "Super Tie" system, developed by RJD Industries for concrete form work, was proposed; however, the study was unable to validate the "Super Tie" system as a feasible post-tensioning



where,

P = Applied lateral load
F = Post-tensioning force

Figure 1.2 Loading Diagram of Post-Tensioned Wall Panel.

system. Steel tendons were utilized for the study to establish a base line for the behavior of the "Formwall" system. Because the panels failed at such a small axial force, additional panels using the nonmetallic tendons and the proposed "Super Tie" system were never constructed and evaluated. "Super Tie," however, was evaluated as described in Section 5.8, Evaluation of Nonmetallic Tendon Anchorage System.

Finally, based on the results and knowledge attained, it is apparent that the "Formwall" concrete masonry units require extensive modifications to pursue any additional testing. These modifications are presented and discussed in Chapter 6, Conclusions and Recommendations. Budgetary and time constraints thwarted any attempt to alter or modify the "Formwall" units for this study.

Chapter 2

CODES AND GENERAL PRACTICES

2.1 Codes and General Practices

The author is unaware of any current nonproprietary information or guidelines governing the use of post-tensioned, dry-stacked masonry. While some European codes, such as British Standard BS 5628: Part 2, provide provisions for prestressed masonry, American codes do not. American building codes, in general, indoctrinate a design methodology based on working stresses and standards of accepted engineering practice. For example, the Building Officials Code Administrators (BOCA) Basic Building Code, the BOCA National Building Code, and the Southern Standard Building Code incorporate the standards of: (1) The American National Standards Institute, Building Code Requirements for Masonry, ANSI A41.1-1953; (2) The American National Standards Institute, Building Code Requirements for Reinforced Masonry, ANSI A41.2-1960; (3) The American Concrete Institute, Building Code Requirements for Masonry Structures, ACI 530-88/ASCE 5-88; and (4) The National Concrete Masonry Association, Specification for the Design and Construction of Load-bearing Concrete Masonry, NCMA TR75-B-1970. The Uniform Building Code outlines particular requirements and procedures, which are similar in scope to the standards listed above, but does not refer specifically to them (Ostag 1986, 8). In purest form, American masonry codes provide the minimum requirements necessary to provide for public health and safety (ACI and ASCE 1990, 2).

While American codes usually include definitions, requirements for materials, and accepted standards of engineering design and analysis, they, in essence, promulgate two methods of design--engineered masonry, commonly referred to as the rational analysis method, and empirical design. While codes are derived and formulated from numerous studies and research, they do not replace sound engineering knowledge, experience, or judgement. For example, requirements more stringent than the Code provisions may be desirable. But under no circumstances are lesser standards permitted (ACI and ASCE 1990, 2).

For purposes of this study, preliminary analyses of the "Formwall" wall panels were based on the rational analysis method, principles of prestressed concrete, and sound engineering judgement. These approaches have been adapted or supplemented, where necessary, in order to properly predict the behavior of the dry-stacked "Formwall" units. Since the preliminary analysis was not validated through testing, this study presents the upper and lower limits used to predict the post-tensioning force required to resist the prescribed loading condition. These limits are presented in Appendix A as best (full composite action between the face shells and ties) and worst (face shells acting independently) case scenarios.

The following sections describe and outline the methodologies used to conduct the preliminary analysis. Actual calculations are presented in Appendix A.

2.2 Methodology of Masonry Design

Under general loading conditions, the minimum requirements of the American National Standards Institute (ANSI A41.2-1960), the American Concrete Institute (ACI 530-88/ASCE 5-88), and the National Concrete Masonry Association (NCMA TR75-B-1970) are recommended as the basis for concrete masonry wall design. All three incorporate the design procedures of working stress analysis in which the determined stresses in the masonry resulting from the effects of all loads and loading conditions do not exceed the prescribed allowable stresses. Moreover, working stress design stems from the concept of straight-line theory, i.e., strains are proportional to stresses. Additional fundamental concepts of working stress design include: (1) plane sections before bending remain plane after bending; (2) all materials are assumed to be homogeneous and isotropic; (3) external forces are in equilibrium with the internal force system; that is, external shears and moments are balanced by the internal resistance; and (4) members are prismatic (Schneider and Dickey 1987, 150-151).

The theory of working stress design was utilized to predict the behavior of the "Formwall" masonry system. However, because of uncertainties with the behavior of dry-stacked masonry and the combatibility relationship between the ties and the "Formwall" face shells, the principles of working stress analysis have been supplemented, where necessary, by sound engineering knowledge and judgement.

2.2.1 Design Methodology for the Nonreinforced Wall Panel

The nonreinforced concrete masonry wall panel was used in this study as a reference base point. The preliminary analysis, which is presented in Section A.1 of Appendix A, was based on the design of nonreinforced masonry as prescribed in NCMA's TR75-B-1970, Specification for the Design and Construction of Load-Bearing Concrete Masonry. The geometric properties--effective area (A) and section modulus (S)--were based upon the criteria of a 6" single wythe wall as determined in NCMA-TEK Bulletin 141A, "Concrete Masonry Section Properties for Design." A copy of this bulletin is included as Appendix D.

2.2.2 Preliminary Design Methodology for the "Formwall" Wall Panel

The preliminary design analysis of the "Formwall" wall panel was based on the theories of engineered masonry, prestressed concrete, and sound engineering knowledge and judgement. As a basis for masonry design, the Specification for the Design and Construction of Load-Bearing Concrete Masonry, NCMA TR75-B-1970, and the effects of combined loading were used. The following discussion, based on NCMA TR75-B-1970, outlines the procedures and principles used by the author in determining the preliminary allowable masonry stresses. Unfortunately, these procedures were not validated by this study since the "Formwall" system failed prior to lateral load testing. Consequently, as aforementioned, the upper and lower limits--composite action (best case)

and the face shells acting independently (worst case)--are presented in order to develop a probable "envelope" of performance of such a system.

1. The allowable compressive stresses were assumed to be based upon the actual compressive strength of the concrete masonry unit (f'_c) as determined and described in Section 3.2.3.1, Unit Strength Method.

2. The allowable axial compressive stress (F_a) was determined from equation (2.1). For the prescribed parameters, best and worst case, the effective thickness (t) was assumed to be the actual width of the "Formwall" unit with ties and the actual thickness of the critical bearing surface of one face shell. Hence, the effective thicknesses were approximately 5" and 1/2"--refer to Table 3.2 for the actual dimensions. For safety reasons, only 25 percent of the allowable axial compressive stress was targeted. If the wall had sustained the post-tensioning force, the allowable axial compressive stress would have been increased incrementally to ultimate failure.

$$F_a = 0.20\, f'_c\, [\, 1 - (h/40t)^3\,], \qquad (2.1)$$

where

F_a = allowable axial compressive stress in psi,

f'_c = compressive strength of masonry unit in psi,

h = effective height in in,

t = effective thickness in in.

The allowable bending stress (F_b) was determined from

$$F_b = 0.33\ f'_c\ (900\ \text{psi maximum}). \qquad (2.2)$$

3. Design load calculations were based upon the effective area (A). The effective area of a "Formwall" unit was based upon the critical bearing surface. For purposes of this study, the effective area was determined using the actual face shell thickness (FST) as described and tabulated in Section 3.2.1.2, Dimensional Evaluation of "Formwall" Units. To satisfy the prescribed parameters, the best case would be two face shells, the worst case, one face shell. In other words, the worst case assumes that one-half of the post-tensioning force will be distributed to each side of the "Formwall" panel.

4. The computed axial stress (f_a) was determined from

$$f_a = P/A, \qquad (2.3)$$

where

f_a = computed axial compressive stress in psi,

P = applied axial force (post-tension force) in lb,

A = effective area in in^2.

The computed axial stress was compared against the allowable axial stress (F_a), determined from equation (2.1).

5. The computed flexural or bending stress (f_b) in compression due to the lateral load was determined from

$$f_b = M/S, \tag{2.4}$$

where

f_b = computed bending stress in psi,

M = applied bending moment in in-lb,

S = section modulus in in^3.

The computed flexural stress was then compared against the allowable bending stress (F_b), determined from equation (2.2). The "Formwall" section modulus, for the prescribed parameters (best and worst case), was determined from

$$S = I/c, \tag{2.5}$$

where

S = section modulus of "Formwall" unit in in^3,

I = moment of inertia ($\Sigma I_o + \Sigma Ad^2$) in in^4,

c = distance from the neutral axis of the two face shells and the extreme fibers in in.

or, worst case where

$$S = 1/6\, L\, t^2, \tag{2.6}$$

where

S = section modulus of "Formwall" unit in in^3,

L = effective length (L) in in.

t = effective thickness (FST) in in,

The assumed geometric properties of the "Formwall" units are listed in Appendix B.

6. Stresses generated from the combined effects of axial and flexural loading must be such that the interaction equation, equation (2.7) is satisfied.

$$f_a/F_a + f_b/F_b \leq 1.0 \tag{2.7}$$

The preliminary design analyses for the "Formwall" panel are presented in Sections A.2 and A.3 of Appendix A.

Chapter 3

MATERIALS

3.1 Concrete Masonry Units

Two types of concrete masonry units were selected for use in this study--hollow load-bearing, referred to as conventional concrete masonry units, and dry-stacked "Formwall" concrete masonry units. Both concrete masonry units were manufactured in accordance with the "Standard Specifications for Hollow Load-Bearing Concrete Masonry Units," ASTM C 90.

3.1.1 Conventional Masonry Units

The conventional concrete masonry units, as manufactured by E. DeVecchis & Sons, Inc., State College, Pennsylvania, are load-bearing, three-core pier units with nominal dimensions of 6" x 8" x 16". The units are Grade N-I, for general use in exterior walls above and below grade, and are comprised of normal-weight concrete with a crushed limestone aggregate. Figure 3.1 shows a typical 6" three-core concrete masonry unit. Certificates for compliance with ASTM standards for the conventional concrete masonry units, masonry cement, and sand are included in Appendix C.

3.1.2 "Formwall" Masonry Units

The "Formwall" concrete masonry units, as developed and manufactured by the National Concrete Masonry Association (NCMA),

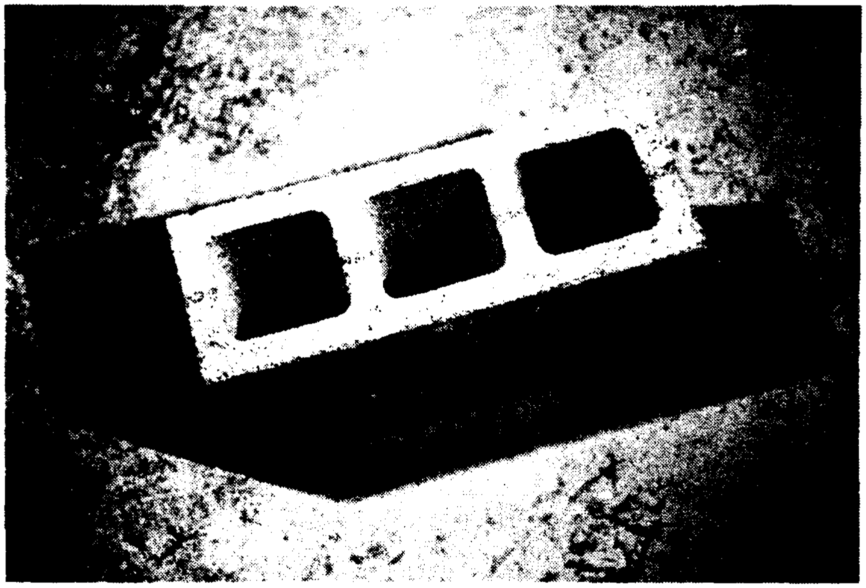


Figure 3.1 Conventional Concrete Masonry Unit.

Herndon, Virginia, consist of two concrete masonry face shells lined with dovetail drain channels. These face shells were initially manufactured for use as horizontal roofing pavers and not vertical load-bearing wall units. Nominal dimensions of a typical face shell are 1" x 8" x 16".

As noted, the "Formwall" units were manufactured as horizontal roofing pavers. In order to configure or stack these units (face shells) into a wall panel, specially fabricated welded wire ties were manufactured. The purpose of the ties is twofold. First, they act as the web of the "Formwall" unit by laterally tying the two face shells together. Second, they act as a key by interlocking the wall's courses.

The ties are manufactured in three sizes (heights)--4", 12" and 16". The first and second courses of the wall panel are tied together using 12" ties, subsequent courses by 16" ties, and the final course with 4" ties. The vertical members (components) of the ties are encapsulated with plastic covers to hold the face shells in place during construction and prevent pull-out or separation of the face shells during loading. Ideally, the ties should fit snugly in the dovetail channels. The ties proposed for this research, however, did not. The plastic covers were nothing more than segments of water hosing. Due to the manner in which hosing is shipped and stored, coiled around a spool, the covers were badly warped and distorted. Consequently, some problems with stacking the face shells and maintaining a constant width along the length of the wall were encountered. Moreover, it raised suspicions concerning the structural integrity of the "Formwall" panel.

Figures 3.2 and 3.3 show a typical "Formwall" face shell and the different size ties and a typical "Formwall" unit with 12" ties.

Figure 3.2 "Formwall" Concrete Masonry
Face Shell and Ties.

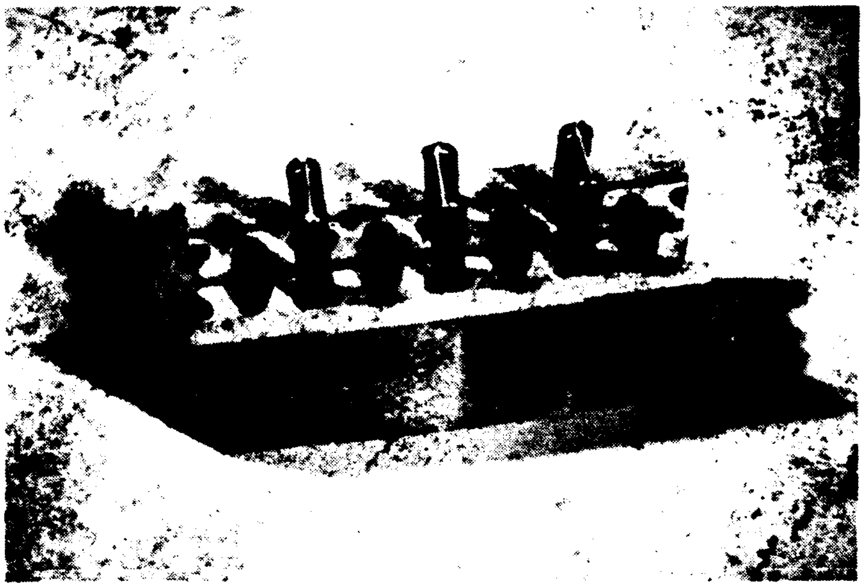


Figure 3.3 "Formwall" Concrete Masonry
Unit with 12" Ties.

3.2 Physical Requirements

Minimum requirements for measurements and dimensions, absorption, unit weight and moisture content, area, volume and density, and compressive strength have been established and codified to control the quality of masonry design and materials. For the purpose of this study, absorption and moisture content were not tested or recorded since they have little or no impact on a mortarless masonry system. While the volume and density could have been determined, they too have been omitted since they provide no relevancy to this study. In the absence of actual tests, procedures for calculating the compressive strength of the "Formwall" units are presented in Section 3.2.3, Compressive Strength of Concrete Masonry Units.

3.2.1 Measurements and Dimensions

In general, "Minimum dimensions for concrete masonry unit face shells and webs insure structural stability of the unit. The minimum face shell thickness provides structural stability of the load-bearing component of the unit. The minimum thickness of the webs limits breakage of the units and provides proper connection and shear transfer between face shells" (NCMA-TEK Bulletin 166 1991). ASTM C 140 specifies the procedures for measurement of dimensions.

Three full size units from the conventional and three face shell units from the "Formwall" lots were selected and measured in accordance with ASTM C 140.

3.2.1.1 Conventional Units

As specified in ASTM C 140, the length (L) was measured on the longitudinal center line of each face; width (W) across the top and bottom bearing surfaces at midlength; and height (H) on both faces at midlength. The face shell (FST) and web (WT) thicknesses were measured at the thinnest point, 1/2" above the mortar-bed plane. Top and bottom face shells were averaged together. The equivalent web thickness (in inches per linear foot of the specimen) was determined by multiplying the sum of the measured thickness of all webs in the unit by 12 and dividing by the length (in inches) of the unit. Average measurements were used to determine the dimensions of the unit. Table 3.1 shows the average measurements for length, width, height, minimum face shell thickness, minimum web thickness for both the interior and exterior webs, and the equivalent web thickness for the three conventional concrete masonry units tested. Each sample met the minimum face shell and web thicknesses and were within the permissible variations as specified in ASTM C 90, "Hollow Load-Bearing Concrete Masonry Units."

3.2.1.2 Dimensional Evaluation of the "Formwall" Units

Preliminary stacking of the face shells indicated possible problems with vertical and horizontal alignment and stability. First, the face shells appeared to be slightly taller at the third points than at the ends (see Figure 3.4--third points measured at e and f). Second, the bearing surface at the top of each face shell was slightly rounded in the transverse direction which allowed the face shell to rotate off center. Hence, additional

Table 3.1 Dimensions of Concrete Masonry Units

Conventional Units

Unit	Avg. Length (L) in.	Avg. Width (W) in.	Avg. Height (H) in.	Avg. Min Face Shell Thickness (FST) in.	Avg. Min Web Thickness (WT) in.		Equiv. Web Thickness in/ft[a]
					Ext Web	Int Web	
1	15.563	5.656	7.563	1.234	1.219	1.250	2.844
2	15.594	5.625	7.625	1.156	1.234	1.219	2.837
3	15.563	5.656	7.562	1.141	1.172	1.250	2.771
Avg. for three units.	15.573	5.646	7.583	1.177	1.208	1.240	2.817

[a]Sum of the measured thickness of all webs in the unit, multiplied by 12, and divided by the length (in inches) of the unit.

measurements other than those specified by ASTM C 140 concerning the effective length, height, and thickness of the "Formwall" face shells were recorded. These additional measurements, which are described below and listed in Table 3.2, provided valuable hindsight to the problems and difficulties experienced in constructing the wall panels. Moreover, the additional measurements, coupled with the results of the dry-stacked conventional wall panel, confirmed the suspicion that the current tolerances specified by ASTM C 90, which were not developed for dry-stacked masonry, are not stringent enough and therefore should not be used for dry-stacked masonry. This is discussed in detail in Chapter 5, Evaluation Criteria.

The length (L) of the "Formwall" units was measured longitudinal along the top, center, and bottom of the face shells; width (W) across the "Formwall" unit (two face shells interlocked laterally by the ties) at both ends and midlength; and height (H) at both ends and midlength of the outer face. Face shell thicknesses (FST) were measured at the "critical" contact or bearing surface at the top of the face shell at both ends, third points, and midlength. Figure 3.4 is a diagram depicting the locations where the measurements of the "Formwall" units were taken. Table 3.2 records these measurements as the actual measurements for length, width, height, and minimum face shell thickness. With the exception of the width, the samples were within the permissible variations as specified in ASTM C 90, "Hollow Load-Bearing Concrete Masonry Units." They did not, however, meet the minimum face shell thicknesses which adversely effected the height to thickness ratio. More importantly, the contact or

Table 3.2 Dimensions of "Formwall" Masonry Units

"Formwall" Units

Unit	Actual Length (L) in.				Actual Height (H) in.				
	a	b	c	Avg.	d	e	g	h	Avg.
1	15.875	15.938	15.875	15.896	7.875	7.875	7.875	7.875	7.875
2	15.938	15.938	15.875	15.917	7.938	7.938	7.906	7.906	7.922
3	15.938	15.938	15.875	15.917	7.875	7.938	7.938	7.938	7.922
Avg. for three units.	15.917	15.938	15.875	15.883*	7.896	7.917	7.906	7.906	7.906*

Refer to Figure 3.4 for location of measurements (i.e. a, b, c, etc.).
*Used in preliminary calculations.

(Table continues on next page)

Table 3.2 (Cont.)

"Formwall" Units

Unit	Actual Width (W) in. d	f	h	Avg.	Actual Face Shell Thickness (FST) in. d	e	g	h	Avg.
1	4.875	5.063	4.938	4.959	0.500	0.594	0.625	0.438	0.539
2	4.813	5.063	4.938	4.938	0.938	0.625	0.563	0.469	0.649
3	4.938	5.125	4.813	4.595	0.938	0.563	0.563	0.938	0.751
Avg. for three units.	4.875	5.084	4.896	4.952*	0.792	0.594	0.584	0.615	0.646*

Refer to Figure 3.4 for location of measurements (i.e. a, b, c, etc.).
*Used in preliminary calculations.

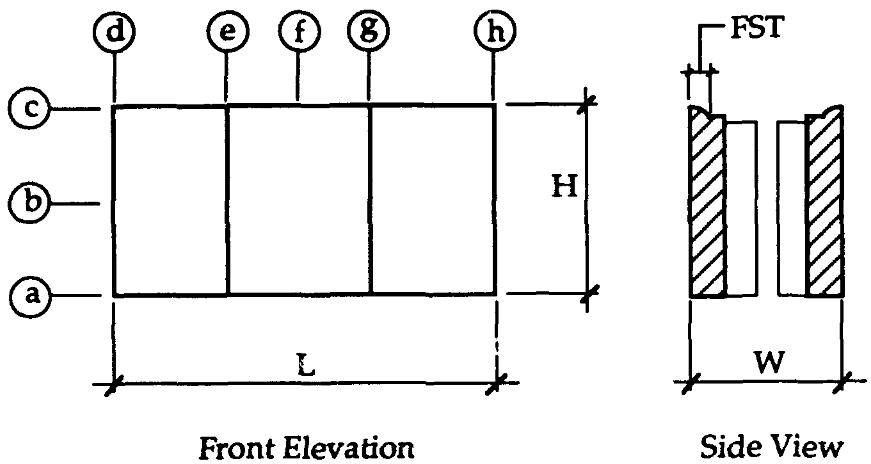


Figure 3.4 Diagram of "Formwall" Measurements.

bearing surface at the top of each face shell was not square (level) but slightly rounded in the transverse direction. This permitted the face shells to rotate slightly which caused problems in stacking the face shells, significantly reduced the effective thickness, and fostered instability in the wall panel. Section 5.6, Evaluation of "Formwall" Panels, describes each of these problems in detail.

3.2.2 Area, Volume and Density

ASTM C 90 does not establish standards for the area, volume, and density of concrete masonry units; however, these properties are important for design calculations and several physical properties tests. The volume and density of the "Formwall" units were not investigated in detail since they had no consequential effects.

3.2.3 Compressive Strength of Concrete Masonry Units

"One of the most important and basic properties used in the design of engineered concrete masonry construction is f'_m, the specified compressive strength of masonry expressed as force per unit of net cross-sectional area (psi)" (NCMA-TEK Bulletin 70A 1986). In determining the specified compressive strength, one of two methods is normally used--Unit Strength Method or Prism Test Method.

The Unit Strength Method, commonly referred to as the assumption method, assumes a value based upon the average compressive strength of the individual units. This method, which is described below, would have been used to determine the compressive

strength of the dry-stacked "Formwall" masonry assemblage and predict the performance of the "Formwall" system had it not failed during testing. Since the actual performance data are not available to validate or determine the factor necessary to modify the predicted f'_m, the Unit Strength Method was not conducted.

3.2.3.1 Unit Strength Method

In accordance with ASTM C 140, "Methods of Sampling and Testing Concrete Masonry Units", three full size "Formwall" face shells would have been selected from the lot and tested in compression. The three face shells would have been capped with gypsum plaster and allowed to cure for 24 hours. After curing, the specimens would have been fitted with 1" steel bearing plates and centered in the testing apparatus. The load would have been applied in the manner specified by ASTM C 140.

In determining the unit's compressive strength, the reported pounds per square inch (psi) would have been divided by the gross cross-sectional area. According to ASTM C 140, "the gross area of a unit is the total area of a section perpendicular to the direction of the load, including areas within cells and within re-entrant spaces unless these spaces are to be occupied in the masonry by portions of adjacent masonry." The compressive strength for the dry-stacked "Formwall" masonry assemblage (f'_m) would have been determined from the compressive strength of the individual units (f'_c).

3.3 Post-Tensioning System

Initially, the post-tensioning system was to utilize nonmetallic tendons with a gripper system developed and manufactured by RJD Industries, Laguna Hills, California. This system, known as "Super Tie" on the market, was not intended to be used for post-tensioning structural members. Experienced and licensed engineers, both in the field and in academia, were suspicious and skeptical about the feasibility of the lock-off mechanism--the "Gripper"--to sustain a post-tensioning force. The initial presumption was that the "Gripper" could not sustain the force. Based on this presumption and time constraints, the nonmetallic tendons and the "Super Tie" anchorage system were substituted with commonly used materials. The nonmetallic tendons were replaced with 1/2" diameter, hot-rolled A 36 steel rods and the "Super Tie" anchorage system with common nuts and washers. The rods were individually threaded, 32" at one end and 3" at the other, to accommodate the nuts. The steel rods were procured from Altoona Pipe and Steel, Altoona, Pennsylvania, and the washers and nuts from Centre Hardware, State College, Pennsylvania. Figure 3.5 shows the components of the post-tensioning system used--a segment of the threaded rod, nuts and washers.

To validate the presumption that the proposed "Super Tie" system would not sustain the post-tensioning force, a test was conducted as described in Section 5.8, Evaluation of Nonmetallic Tendon Anchorage System. Much to the author's surprise, the system could have been used. It sustained an ultimate prestressing force of approximately 9,420 pounds. The post-tensioning force required for the "Formwall" panel was 3,500 pounds per tendon or approximately 37 percent of the "Super Tie"

capacity. This is not to state, however, that the "Super Tie" system is a solution to post-tensioning; but rather, it disproves the initial presumption and justifies additional testing. Figure 3.6 shows the components of the proposed nonmetallic post-tensioning system--a segment of the nonmetallic tendon, "Gripper," and "Rock Grip."

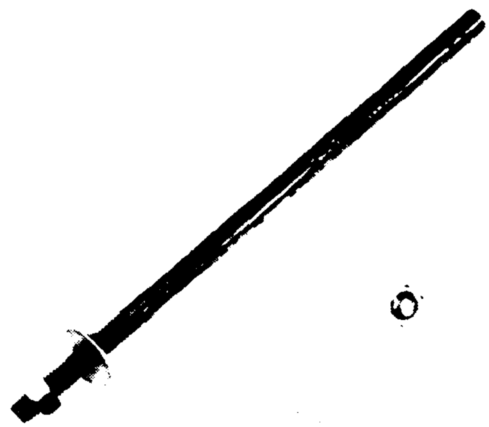


Figure 3.5 Components of the Steel Post-Tensioning System Used.

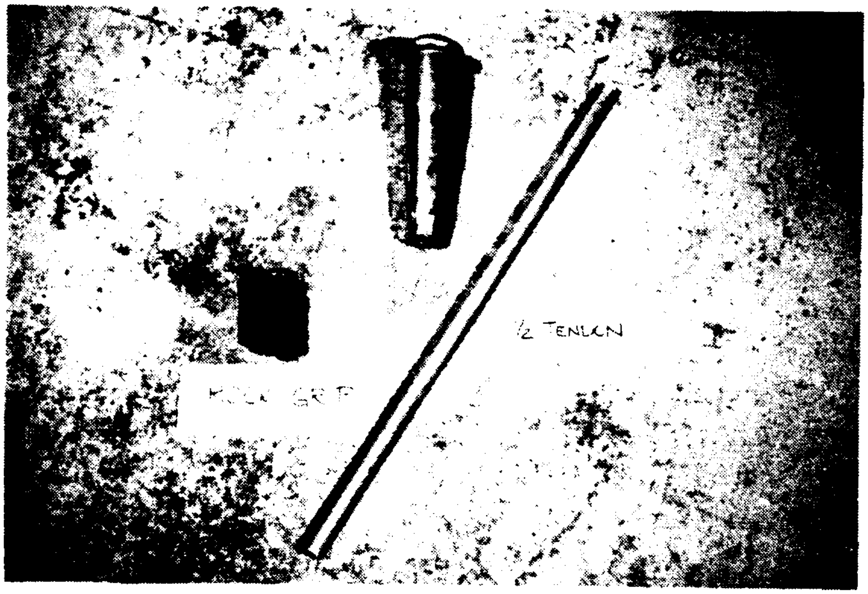


Figure 3.6 Components of the Proposed Non-metallic Post-Tensioning System.

Chapter 4

WALL PANELS

4.1 Description

Four masonry panels, measuring 4′ x 8′-8″ and supported vertically at 8′ on centers, were constructed, evaluated, and tested. Two of the panels were constructed using NCMA's dry-stacked "Formwall" concrete masonry units while the other two consisted of conventional 6″ concrete masonry units. All four panels were constructed using a running bond.

4.1.1 Conventional Panels

The two conventional wall panels were constructed and tested to establish a base line for comparison and provide validity to the testing apparatus and equipment. Both consisted of 39 (13 courses) three-core, concrete masonry pier units and were supported vertically at 8′ on centers. The first panel was nonreinforced and constructed with Type "S" mortar, the other was dry-stacked. The primary purpose of the dry-stacked conventional panel was to establish a comparison and substantiate the findings that the "Formwall" units were unstable.

4.1.2 "Formwall" Panels

The "Formwall" panels consisted of 39 (13 courses) of dry-stacked "Formwall" concrete masonry units. As described in Section 3.1.2, each unit consisted of two face shells tied together laterally by specially fabricated metal ties designed and manufactured by NCMA. (Previous

figures, Figures 3.2 and 3.3, showed a typical "Formwall" face shell and unit with the specially fabricated ties.) The panels were reinforced concentrically and post-tensioned using two 1/2" diameter steel tendons. The post-tensioning force was introduced using a 10,000 pound per square inch (62,800 pound) center-hole, twin cylinder ram and a hand operated hydraulic pump. Figure 4.1 shows the ram and pump. A complete description on how the post-tensioning force was introduced and locked off is provided in Section 4.3, Post-Tensioning of "Formwall" Panel.

4.2 Construction of Wall Panels

The mortarless, dry-stacked "Formwall" panels and the dry-stacked conventional panel were constructed by the author to better understand and evaluate the constructibility of a dry-stacked system. The mortared, nonreinforced panel was constructed by an experienced mason. Each panel was constructed in place within the test frame. The sequence of construction is as follows.

4.2.1 Construction of Nonreinforced Panel

The nonreinforced panel was constructed in conventional fashion using Type "S" mortar. The courses were laid in full face shell bedding. The joints, approximately 3/8" in thickness, were struck flush. The first course rested freely on a 1/2" steel plate 4' long on which was welded a half of a 3" steel pipe (refer to Figure 4.2). This plate, referred to as a "rocker plate," reduced the friction between the concrete masonry panel and the concrete slab upon which it sat, helping to provide pinned end

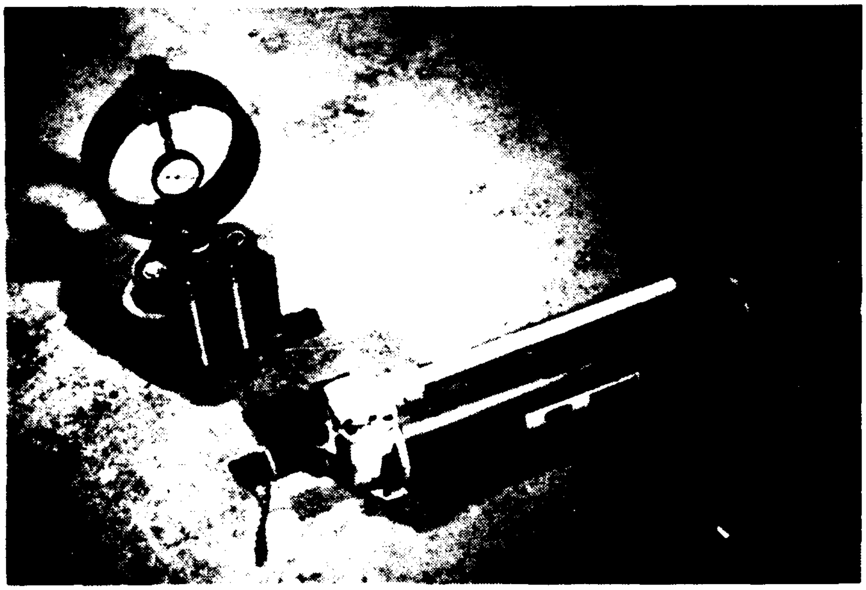


Figure 4.1 Twin Cylinder Ram and Hydraulic Pump.

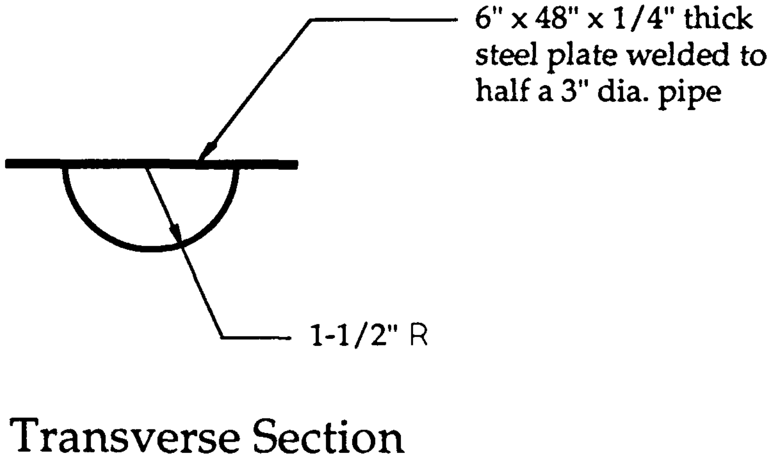


Figure 4.2 Steel Rocker Plate.

conditions. The rocker plate was replaced by an inverted channel for subsequent testing, i.e. the "Formwall" and the dry-stacked conventional panels.

4.2.2 Construction of "Formwall" Panels

The "Formwall" panels were built on an inverted, continuous 4′ steel channel--C 9 x 15. Prior to constructing the panels, the post-tensioning tendons were positioned in pre-drilled holes, 16″ on center. The tendons were secured to the underside of the channel, which was reinforced with a 1/4″ steel bearing plate, with common 1/2″ nuts and washers. The tendons were temporarily secured at the top to ensure concentric loading. With the tendons in place, the "Formwall" units were stacked face shell by face shell around them.

The first course was tied together using the 12″ ties. Initially, the ties were spaced at 8″ on center ("Formwall" Panel 1) but were changed to 4″ on center ("Formwall" Panel 2) to assist in stability and to facilitate face shell alignment. The ties are designed to extend 4″ above or below the top of the face shells, depending on the course, to key adjoining courses (see Figure 4.3). After the first course, subsequent courses were tied together using 16″ ties with the final course using 4″ ties. Unfortunately, the ties were not flush with the top of the final course and blocking had to be installed to provide an even surface for the top bearing plate. The blocking and bearing plate were constructed from 2″ x 6″ pressure-treated lumber. The blocking spanned across the face shells and fit in between the ties and post-tensioning rods. With the bearing plate in position, the wall panel was post-tensioning. No curing time was required.

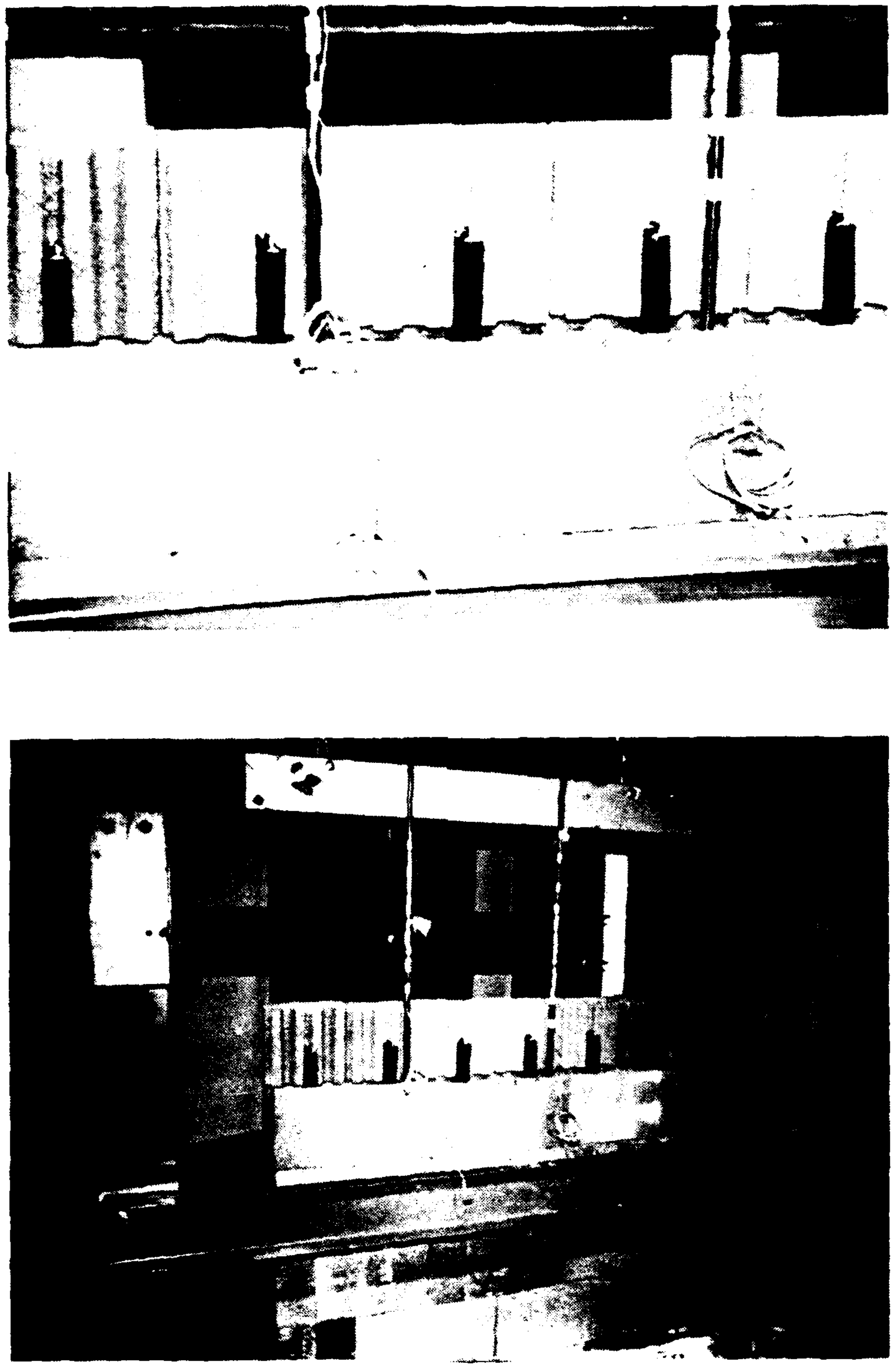


Figure 4.3 "Formwall" Panel 1 During Construction.

The construction time was approximately three hours for one individual. This is not however, a good measuring stick of the construction time. The wall was built with extreme care to ensure stability, proper alignment, and prevent damage to the strain gages attached to the post-tensioning tendons.

4.2.3 Construction of Dry-Stacked Conventional Panel

The dry-stacked conventional panel was constructed on top of the same inverted channel as the "Formwall" panels. Each concrete masonry unit was carefully stacked one on top of the other until all 39 units (13 courses) were in place. The panel was capped with a 2" x 6" pressure-treated lumber bearing plate. Like the "Formwall" panels, the dry-stacked conventional panel required no curring time.

4.3 Post-Tensioning of "Formwall" Panel

The post-tensioning force was introduced using a center-hole, twin cylinder ram equipped with a compression ring. The ram was positioned on top and at the center of the panel and rested freely on the pressure-treated bearing plate. At each tendon, a second bearing plate, a pre-drilled 6" x 6" x 1" thick steel bearing plate, rested freely on the 2" x 6" wood plate. A common 1/2" washer and nut were threaded down each tendon and hand tightened against the steel bearing plates. Next, a 2" x 2" x 1/4" thick structural tube, with pre-drilled holes, was lowered over the tendons and positioned on top of the compression ring and two adjustable lally columns. The lally columns were prepositioned on either side of the steel

tendons. A second 1/2" washer and nut were threaded down to and hand tightened against the structural tube. The post-tensioning force was applied by a hand operated hydraulic pump and ram. The lally columns held the force while the nut on each tendon was tightened. This was done to prevent prestress loss caused by hydraulic bleeding. Figure 4.4 shows the post-tensioning system in place and ready for use.

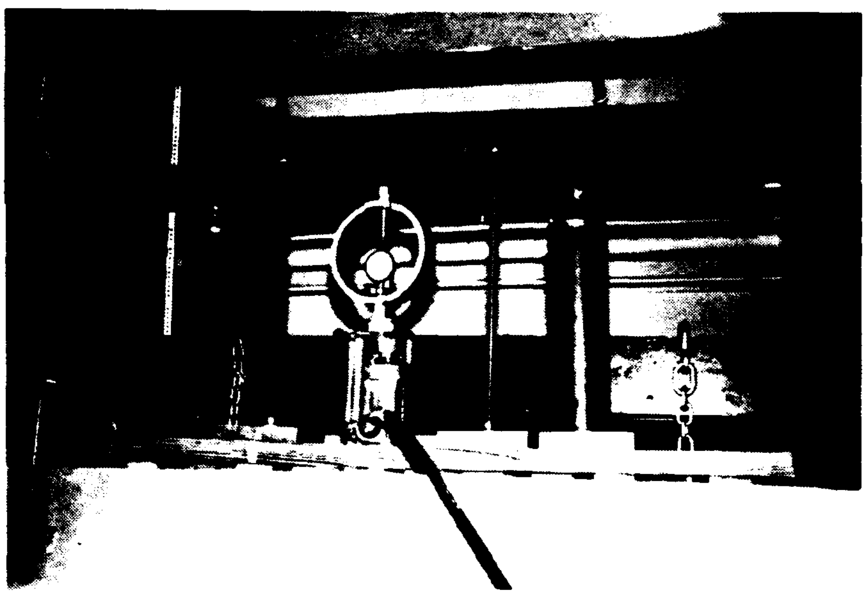


Figure 4.4 Post-Tensioning System on Top Wall Panel.

Chapter 5

EVALUATION CRITERIA

5.1 Introduction

Since "Formwall" is a new and untested masonry system (product), a logical and methodical approach was taken to evaluate its performance. First, a nonreinforced masonry panel was constructed and tested in flexure to develop a base line of comparison, provide validity to the testing apparatus and equipment, and provide the researcher hands-on experience. Second, adjustments were made to the testing apparatus and equipment based on the results of the nonreinforced panel. Third, qualitative data were collected during the construction of the "Formwall" panel to develop an understanding about the product and dry-stacked masonry in general. And fourth, the "Formwall" panel was post-tensioned incrementally to develop the strength required to resist a prescribed lateral load.

As indicated in Section 1.3, Limitations, the "Formwall" panel buckled during the application of post-tensioning at a relatively low prestressing force. While this might have undermined tests regarding the flexural capacity of the system, an abundance of information was collected concerning inherent problems with dry-stacked masonry in general. This information along with descriptions of the test frame, instrumentation used to monitor behavior, testing procedures, and problems encountered are presented in the following subsections.

5.2 Test Frame

Figure 5.1 depicts the loading condition used for this study. The frame was constructed from steel wide flange sections donated by Milton Steel, Inc., Milton, Pennsylvania. The frame was configured such that the specimens could be analyzed and tested as simple supported elements. To aid in simulating pinned end conditions, a 1/2" diameter steel rod was attached along the face of the two horizontal W14 x 30s acting as reactions. The rods reduced the contact surface of the reactions and permitted rotation; thereby, simulating pinned end conditions. Within the frame, a load transfer system (spreader beam), consisting of three W6 x 9s, was assembled to distribute the applied concentrated lateral load as two equal point loads symmetrically placed. This loading pattern best simulates a uniformly distributed load.

5.3 Loading Procedures

Lateral loading was applied to the wall panels by a hand operated scissors jack. The loads were applied in 10 pound increments after an initial loading of 105.2 pounds. The significance of the 105.2-pound load is that this was the smallest force the compression ring, being used to measure the applied force, could measure. The loads were transferred from the scissors jack to third points along the wall panel by the spreader beam.

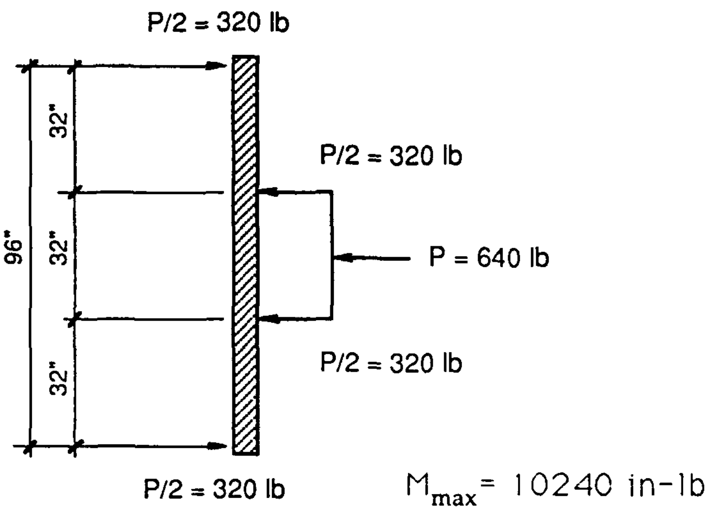


Figure 5.1 Loading Diagram.

5.4 Instrumentation

Each wall panel was monitored for applied loads and horizontal deflections. Additionally, each panel was monitored for characteristics particular to the type and purpose of the panel. For example, the post-tensioning tendons were monitored for prestress loss using electronic strain gages.

The following subsections describe the instruments used by wall panel type.

5.4.1 Instrumentation for the Nonreinforced Panel

The nonreinforced panel was constructed and tested to validate the testing apparatus and equipment. This panel was monitored for tensile strains in the concrete masonry units and horizontal deflection using a Whittemore extensometer and potentiometers.

To monitor the tensile strains in the concrete masonry units the tensile face of the panel received six gage targets spaced 5″ apart vertically along the center line of the panel. The 5″ spacing provided four readings as measured by the Whittemore extensometer. The Whittemore extensometer has a least reading of 1×10^{-5} inch per inch.

Lateral deflections, were measured at the center of the panel and two places vertically on either side of center. Readings were taken electronically with potentiometers. The potentiometers provided readings to an accuracy of 0.02″.

5.4.2 Instrumentation for the "Formwall" Panels

As a result of difficulties experienced with collecting data from the nonreinforced mortared panel coupled with the behavior differences of a mortarless and mortared masonry system, changes were made as to what and how deformations were to be monitored. For example, there was no distinct advantage in measuring the strains in the dry-stacked masonry. A more practical and useful measurement would be the prestress loss in the post-tensioning tendons. Consequently, targets were not placed on the tensile or compression face of the wall panel. Another example is with the data acquisition system. The author experienced difficulties in collecting consistent data with the data acquisition system; hence, deflection readings were taken mechanically using dial gages. Although changes were made after testing the nonreinforced panel, these changes had no relevant consequences on the validity of testing.

The "Formwall" panels were set up to monitor lateral deflection and post-tension (prestress) loss using dial gages and electronic strain gages, respectively. The dial gages were positioned at the center of the panel and two places vertically on either side of center. The gages provided readings to an accuracy of 0.001".

Each tendon received three electronic strain gages mounted at the center of the tendons (mid-height for the panel). All electronic strain gages were applied in accordance with the manufacturer's recommendations. The strain gages were used to verify the post-

tensioning force applied, measured by a compression ring, and post-tension loss.

5.4.3 Instrumentation for the Dry-Stacked Conventional Panel

This panel was subjected to an applied axial force to substantiate the finding that the "Formwall" wall panel was geometrically unstable. Therefore, the panel was monitored solely for the axial force using a compression ring mounted to a hydraulic jack which pushed against the top of the testing frame.

5.5 Evaluation for the Nonreinforced Panel

As previously stated, the nonreinforced panel was constructed and tested to establish a base line of comparison and provide the researcher with hands-on experience. Hence, analytical computations were performed prior to conducting the test to predict the magnitude of the lateral load which would crack (fail) the panel and determine the deflected shape. (The lateral load computation is presented in Section A.1 of Appendix A). In addition, the data acquisition system, potentiometers, and Whittemore extensometer were calibrated and tested.

Overall, the panel behaved as expected and proved to be a successful indicator of performance. It cracked at the location of the maximum moment and at a load greater than calculated--actual load 334 pounds, theoretical load 302 pounds. Nevertheless, factors were discovered which effectuated modifications before testing the "Formwall" panels. These factors are discussed below.

5.5.1 Problems Encountered During Evaluation of Nonreinforced Panel

The first problem was with the wall panel itself. The rocker plate, upon which the wall panel was constructed, rotated slightly during the curing time. Consequently, the top and bottom of the panel were not in immediate contact with the supports. An initial force was required to seat the panel in place causing a hair line crack to develop between the first and second courses. The load which caused the crack was unattainable since the smallest load the compression ring (being used to measure the applied load) could measure was 105.2 pounds. The hairline crack occurred well before 105 pounds. Second, the Whittemore Extensometer, used to measure the tensile strains in the outer fibers of the concrete masonry units was not sensitive enough. One increment on the dial gage was equivalent to 18 psi. The allowable tensile stress was 24 psi. Consequently, no readings were recorded. Third, the potentiometers were not adequately secured to the testing frame. As the load was applied and the specimen rotated to seat itself against the frame, the potentiometers shifted. Moreover, the potentiometers were observed to "stick" on occasion. These two problems proved to be significant as they contributed to the inconsistent data readings by the data acquisition system. Fourth, the data acquisition system had a limited number of channels. Of the five dedicated to potentiometers, only four were working. Finally, the spreader beam was positioned according to vertical measurements at the location where the beams were being suspended rather than at the final location. As the load was applied, the beams shifted upward and changed the

loading condition. The preliminary calculations were revised accordingly to reflect the new loading condition.

5.5.2 Evaluation Results of the Nonreinforced Panel

While no quantitative data were collected other than the applied load and visual observations, the test proved to be useful. First, the wall panel behaved as expected. It cracked along the tensile face at the location of the maximum moment and at a load greater than calculated--wall panel failed at 334 pounds compared to the calculated load of 302 pounds. Second, all problems encountered were correctable. The rocker plate was replaced with an inverted steel channel, the potentiometers were replaced with fixed (stationary) dial gages, and the spreader beam was located in accordance with its prescribed height.

5.6 Evaluation of "Formwall" Panels

Testing of the "Formwall" wall panels included construction and application of the post-tensioning force. Since the wall panels experienced an instability problem, which resulted in buckling during post-tensioning, the "Formwall" panels were not tested in flexure.

5.6.1 Evaluation Results of "Formwall" Panel 1

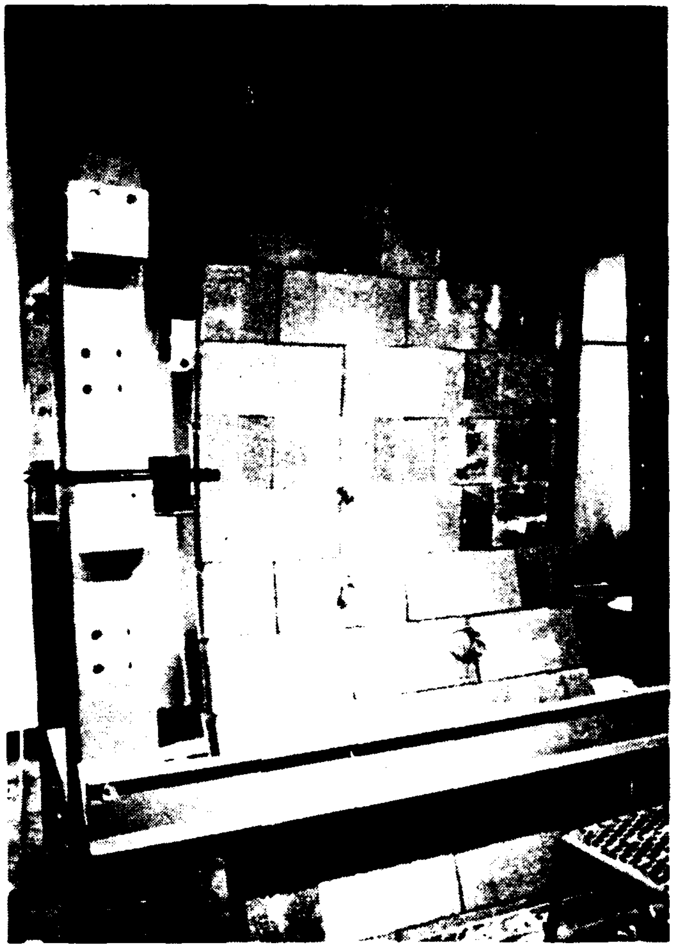
The first "Formwall" panel was carefully constructed in the manner described in Section 3.2.1, "Formwall" Panels. Prior to placing the eleventh course, or at a height of approximately 7′, the wall panel began to

lean inward, in relation to the test frame, and fell back against the spreader beam. The wall was unbraced during construction.

Figure 5.2 shows two veiws of Wall Panel 1 in its collapsed state. Note that the ties, which act as the face shells web, slipped in relation to the face shells and went along for the ride (Figure 5.2 (b)). The failure mode experienced reinforced the predicted concept that the wall system would not behave with composite action. Examination of the ties and face shells after dismantlement revealed no apparent damage whatsoever to either. These materials, however, were not reused for future testing.

5.6.2 Evaluation Results of "Formwall" Panel 2

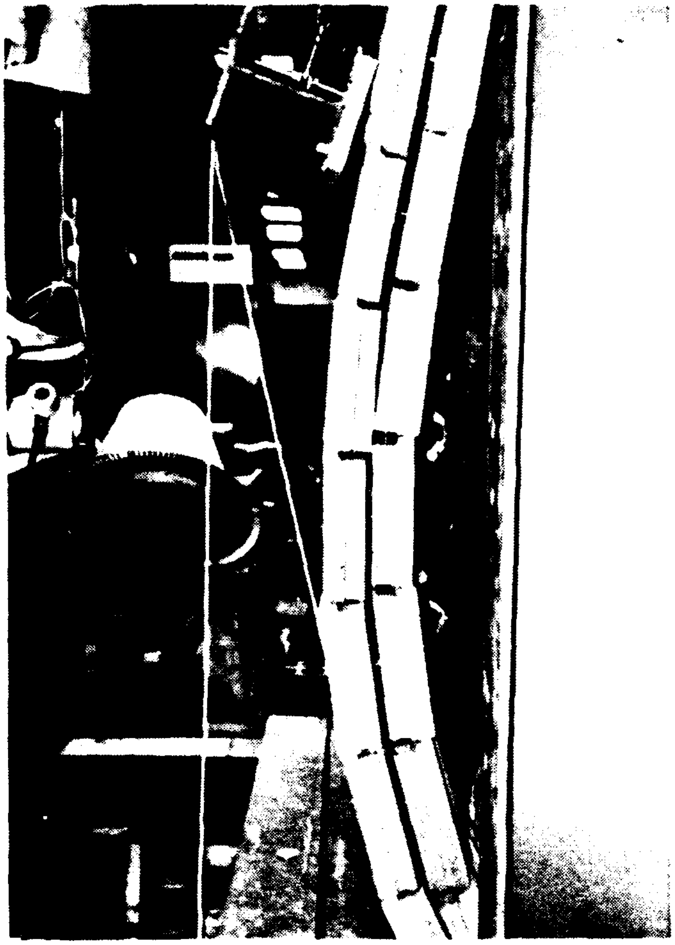
The second "Formwall" panel was also carefully constructed in the manner described in Section 3.2.1; however, temporary bracing was used at mid height--48". The wall panel was completed and pinned against the top support (against the 1/2" diameter rod attached to the W14 x 30) by wood blocking--simulating floor joists framing into the wall panel. Prior to post-tensioning, the author visually inspected the wall and made the following observations: (1) the wall panel was not plumb vertically or horizontally, (2) large separations existed between the vertical and horizontal joints, and (3) the wall panel appeared to be unstable (shaky). Each of these observations is discussed in detail below.



(a) Wall Panel 1 collapsed under its own weight at approximately 7' - 0". No bracing was provided during construction.

Figure 5.2 "Formwall" Wall Panel 1 in Collapsed State.

(Figure continues on next page)



(b) Side view showing how the ties slipped in relation to the face shells. No damage was observed to either the face shells or ties.

Figure 5.2 (Cont.)

5.6.2.1 Wall Alignment and Joint Separation

The wall panel was not aligned vertically or horizontally. Problems with vertical alignment stemmed from the imperfections with the concrete masonry face shells. In other words, the face shells, as indicated in Table 3.2, were not manufactured uniformly. Moreover, the majority, if not all of the units, had round, porous edges which fostered a poor bearing surface and a tendency to rotate. This condition prevented bearing contact between some of the face shells and reduced the bearing surface of others to a single line of contact. In other words, the average linear bearing contact between face shells for any given course on the tensile face of the panel measured to be 28.32" in lieu of the full 48" panel width. In other words, only 59 percent of the tensile face of the wall panel had full contact between the face shells. And of those face shells, there was a very high probability that the bearing surface was nothing more than a single line of contact. This significantly affected the height-to-thickness ratio which reduced the allowable compressive stress and increased the probability of buckling, which is what happened. Vertical separations were not recorded; however, gaps were detected up to 1/4".

Figure 5.3 shows "Formwall" Wall Panel 2 prior to post-tensioning. The white chalk lines indicate full horizontal bearing contact between the face shells. Figure 5.4 shows the maximum horizontal and vertical separations observed between the face shells on the tensile face of the wall panel prior to post-tensioning. As indicated, vertical separations were as large as 1/4" and horizontal separations as much as 7/32". At several locations, as evident in Figure 5.4, the ties were visible and at other locations the author could see "daylight" from the opposite side.

Figure 5.3 "Formwall" Panel 2 Prior to Post-Tensioning.

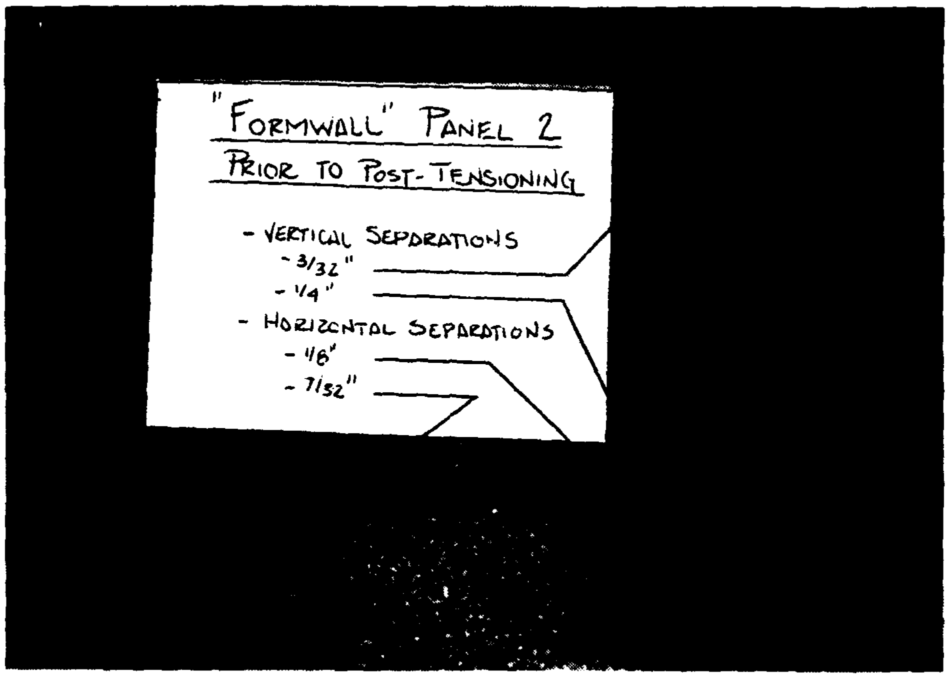


Figure 5.4 Separations Between Face Shells on Tensile Face of Wall Panel 2 Prior to Post-Tensioning.

Problems with horizontal alignment stemmed primarily from the specially fabricated ties which keyed the courses and controlled wall width. It is the author's opinion, based on his experience of stacking the wall panels, that the ties manufactured for this particular project were not fully compatible with the dovetail channels due to irregularities in size and uniformity of the plastic hosing. For example, the plastic hosing, which sheathes the metal tie, was badly warped due to the nature in which the material is stored and shipped--water hosing is tightly coiled. The warped hosing prevented a tight, "glove-like" fit within the dovetail channels. Consequently, it was difficult to stack the face shells straight and plumb. Moreover, similar size ties were not the same size in height or width. At the top of the wall panel, where the ties needed to be flush with the top course in order to place the bearing plate, the ties protruded any where between half an inch to two inches. This resulted in adding wood blocking, spanning across the face shells and in between the ties, prior to placing the bearing plate. More importantly, however, the difference in widths promoted the rotation of the face shells about their bearing surface which affected the stability of the wall panel.

5.6.2.2 "Formwall" Wall Panel Stability

As the research continued, it became more and more evident that the "Formwall" wall panel was unstable. Unlike a conventional block, which gets its stability from the integral web, the "Formwall" units rely on a narrow and imperfect contact surface coupled with the specially

fabricated ties. As previously stated in Section 5.6.2.1, the imperfections in the bearing surface fostered rotation between the face shells. The ties, which act as the "Formwall" unit's web, provided little to no resistance against this rotation. Consequently, the "Formwall" wall panel was geometrically unstable and unable to sustain the post-tensioning force. This observation was substantiated by the dry-stacked conventional concrete masonry wall panel described in Section 5.7, Evaluation of the Dry-Stacked Conventional Panel, which did not exhibit similar behavior.

5.6.2.3 Post-Tensioning the "Formwall" Panel

After visually inspecting the "Formwall" panel and zeroing the instrumentations, the author post-tensioned the wall panel. The initial axial load was 105.2 pounds as measured by the compression ring. It was determined that loads would be applied in 100-pound intervals (plus or minus 10 pounds) up to 5,400 pounds or 2,700 pounds per tendon. While preliminary calculations, included in Appendix A, indicated that a post-tensioning force of this magnitude--5,400 pounds--assumes full composite action between the face shell and tie (which is extremely optimistic and very unlikely), it does provide a margin of safety for this untested masonry system. In other words, to assume noncomposite action would require a post-tensioning force of 143,684 pounds which far exceeds the allowable axial stress (F_a) as determined from equation (5.1).

$$F_a = 0.20 f'_c [1 - (h/40t)^3] \qquad (5.1)$$

At a post-tensioning force of approximately 200 pounds, the wall panel began to to buckle. Without any additional loading, the wall panel continued to deflect and buckle until the panel experienced double curvature--an "S" shape. Figure 5.5 shows the wall panel in its deflected shape. Data representing the deflection of the wall panel and strain in the tendons were unattainable due to the unexpected and sudden failure.

5.7 Evaluation of the Dry-Stacked Conventional Panel

The dry-stacked conventional panel was evaluated to validate the argument that the "Formwall" panel was unstable. As mentioned above, the conventional concrete masonry units met the minimum face shell and web thicknesses and were within the permissible variations as specified in ASTM C 90, "Hollow Load-Bearing Concrete Masonry Units." Nevertheless, the conventional wall panel experienced a similar problem in the area of developing full contact between bearing surfaces as did the "Formwall" panel. It was determined that this was due to the porous nature of concrete masonry. Because conventional concrete masonry units are relatively flat (have level surfaces), the conventional panel did not experience as severe a problem in developing contact as the "Formwall" panel.

For fair reason, the dry-stacked conventional wall panel was able to sustain an axial load of over 550 pounds. Loading would have continued; however, the test frame was not bolted to the slab. The 550-pound load was viewed as adequate assurance that the conventional panel had sufficient structural integrity for post-tensioning as compared to the

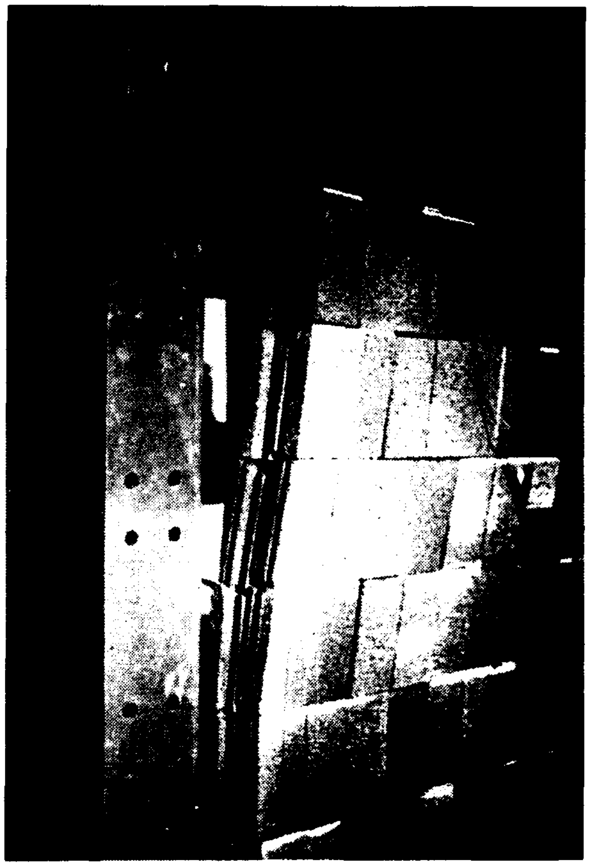


Figure 5.5 "Formwall" Panel 2 After the 200-Pound Axial Load Was Applied.

"Formwall" panel. While additional testing is warranted, this simple test reveals that the "Formwall" units, in their present configuration, lack the stability required for a load-bearing masonry wall. For this reason, subsequent work was concentrated in the area of suggesting a more stable geometric block shape for further testing. See Chapter 6, Conclusions and Recommendations.

5.8 Evaluation of Nonmetallic Tendon Anchorage System

As previously stated, the proposed post-tensioning system utilized nonmetallic tendons and an anchorage system, known as "Super Tie," as developed and manufactured by RJD Industries. In order to determine whether or not the "Super Tie" system could be employed as a post-tensioning system, a test was conducted using two of the "Grippers" and a 30" segment of the nonmetallic tendon. Initial research indicated that this type of anchorage system has a tendency to slip under significant loads.

The tendon was placed in a center-hole, twin cylinder ram which was connected to a hand operated hydraulic jack. A pre-drilled, 1" steel bearing plate was positioned at either end between the ram and "Gripper." The entire assembly--ram, tendon, bearing plates, and "Grippers"--was placed on concrete masonry units as shown in Figure 5.6.

Loads were applied in 500 pounds per square inch (psi) or 3,140 pound increments. The tendon sustained an ultimate load of approximately 9,420 pounds. An exact measurement was not possible due to the increment readings of the dial gage--500psi. This value, however, is well in excess of what was predicted to be necessary for the "Formwall"

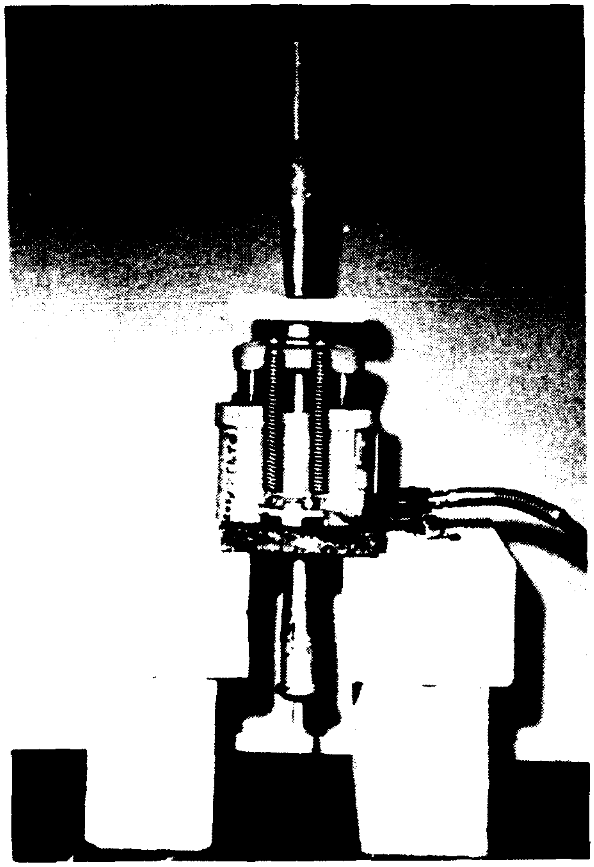


Figure 5.6 Test Assembly of the "Super Tie" System Prior to Test.

system. Hence, the "Super Tie" system proved to be useful for moderate prestressing forces. Some practical use problems such as the method of introducing the post-tensioning force, time dependent prestress loss, and cost remain and must be resolved if this anchorage system is to be considered for actual field use. Figure 5.7 shows the failed tendon.

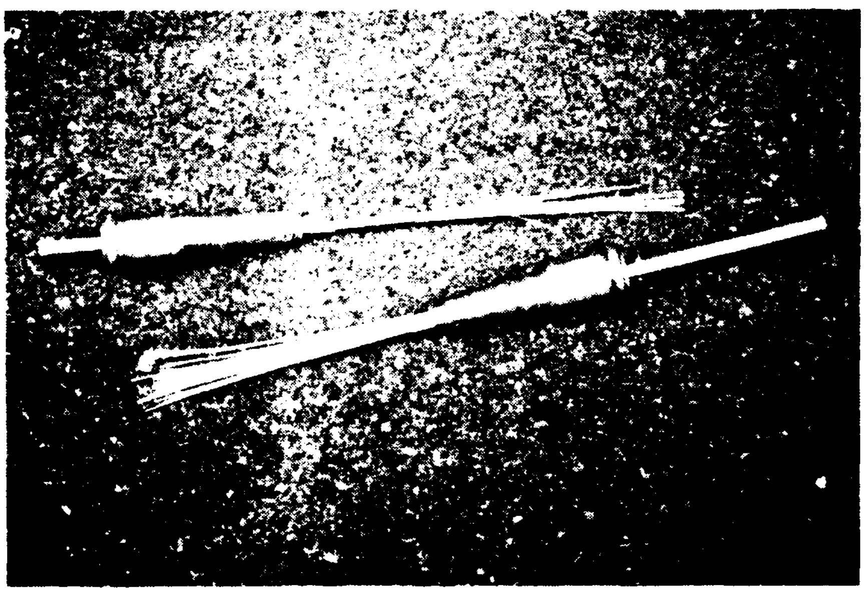


Figure 5.7 Nonmetallic Tendon after Test.

Chapter 6

CONCLUSIONS AND RECOMMENDATIONS

This study examined the constructibility, feasibility, and overall performance of a new and untested post-tensioned, dry-stacked concrete masonry wall system--know as "Formwall"--as proposed by the National Concrete Masonry Association. The evaluation was conducted through mathematical predictions, wall construction, and experimental testing. To substantiate the findings, comparisons were made between the dry-stacked "Formwall" system, a conventional nonreinforced (plain masonry) wall panel with Type "S" mortar, and a dry-stacked wall panel using conventional 6" concrete masonry units. Based on the overall evaluation of this study, the proposed "Formwall" system has virtually no structural capacity due primarily to stability issues stemming from problems related to the geometric shape of the face shells. Notwithstanding, it is the author's opinion that there is merit in pursuing the development of a dry-stacked masonry system. It is feasible that dry-stacked masonry, if perfected, could offer significant savings in labor and material costs. Obviously, ultimate savings in the latter include some economic factors which are unforeseeable at this time--i.e. technological developments and supply and demand.

6.1 Constructibility and Feasibility

Dry-stacked masonry is relatively easy to construct, requires less man-hours, and is more manageable than conventional masonry using mortar and grout. In fact, the author, who possesses no masonry

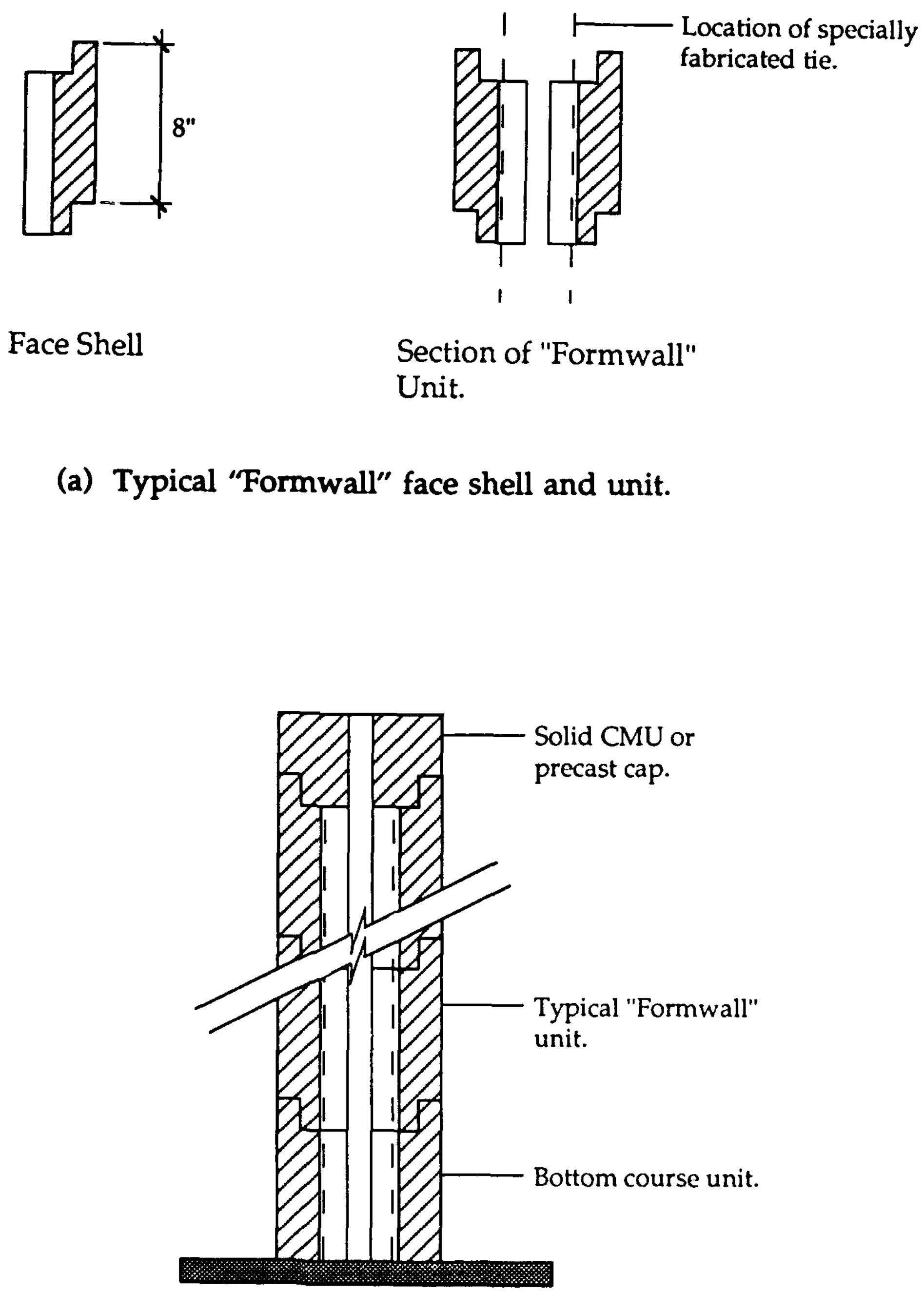
construction skills whatsoever, was able to construct and post-tension the "Formwall" panel in less than three hours and the conventional dry-stacked panel in less than one hour. As previously stated in Section 4.2.2, the time required to build the "Formwall" panel included additional time to ensure that the panel was as stable as possible and that care was taken not to damage to sensitive strain gages affixed to the post-tensioning tendons. In other words, the time required to construct the "Formwall" panel for this study is not a good indicator of actual (field) construction time. Notwithstanding, the dry-stack panels--"Formwall" and conventional--were constructed and tested in one day, while the nonreinforced panel, including curing time, took over a week.

While the dry-stacked conventional panel was easier and faster to construct than the "Formwall" panel, the face shell concept of the "Formwall" system provides an easier and more efficient method of stacking concrete masonry in a running bond around post-tensioning tendons. Moreover, they are lighter in weight and more manageable. However, as experienced in this study, before "Formwall" can be realized as a structural element, modifications to the units themselves and the specially fabricated ties must be made to ensure structural stability and overall wall assembly integrity.

Figure 6.1 schematically depicts a proposed face shell which could provide the stability required. (Details including the dovetail channels to accommodate the specially fabricated ties are left open at this time. It is worth mentioning, however, that the ties, presented in a subsequent section, are recommended to be "form fitting.") The proposed face shell increases the face shell thickness (FST) and squares (levels) all bearing

surfaces. If practical, the the bearing surfaces should be "smooth." Dimensions (which are purposely omitted in Figure 6.1) should be based on the requirements specified by ASTM C 90; however, additional testing is recommended to determine if these requirements are adequate. In addition to thickening the face shell, the proposed face shells are keyed to enhance the bearing surface and minimize rotation which fostered buckling of the "Formwall" wall panel. As indicated and depicted in Table 3.2 and Figure 3.4, the "Formwall" face shells used in this study had less than 1/2" of bearing surface which was slightly rounded. Thus, as the face shells were stacked, they had a tendency to rotate, causing "kinks" longitudinally (vertically). Consequently, the wall panel was geometrically unstable and collapsed when subjected to the post-tensioning force. The proposed face shell, with a proper tie, should preclude the instability problems experienced by the "Formwall" masonry system.

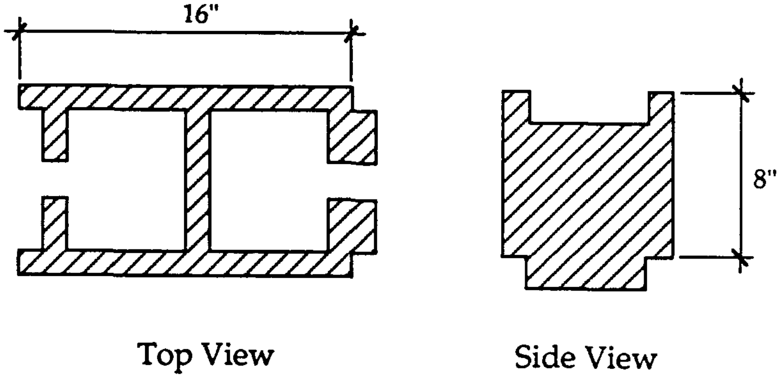
An alternative to the face shell concept is depicted in Figure 6.2--a dry-stack concrete masonry unit. This proposed unit is similar to a conventional concrete masonry unit with the exception to its geometric shape. The unit has been carefully designed to enhance stability and allow its use in running bond patterns. As depicted, the blocks have been keyed to interlock subsequent courses to enhance stability. In addition, the ends provide a 1" opening to allow the unit to be used in a running bond with pre-positioned post-tensioning tendons. Either of the of units proposed--the face shell or dry-stack concrete masonry unit--could bring dry-stack masonry into fruition.



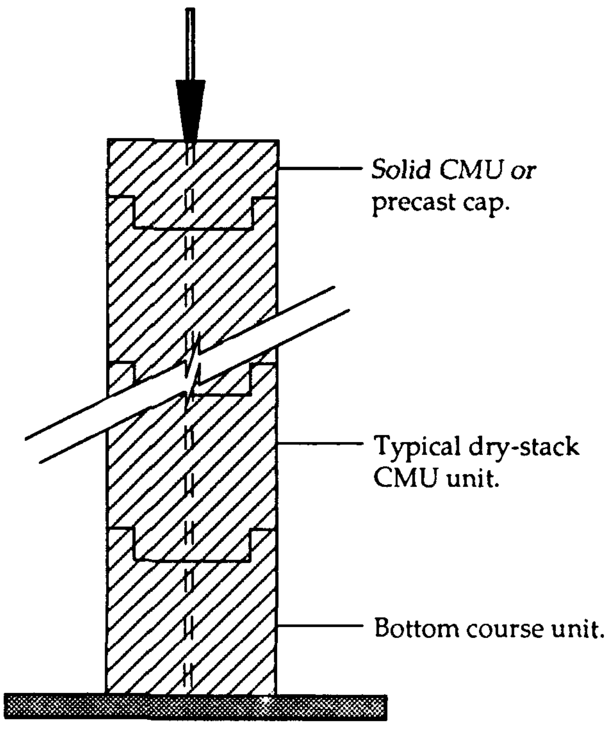
(a) Typical "Formwall" face shell and unit.

(b) Wall section.

Figure 6.1 Proposed "Formwall" Unit.



(a) Typical dry-stack concrete masonry unit.



(b) Wall Section.

Figure 6.2 Proposed Conventional Dry-Stack Concrete Masonry Unit.

6.2 Specially Fabricated Ties

The ties proposed for this study had several inherent problems. First, they were not uniformly fabricated. Similar size ties were not the same size, the plastic sheathing was warped and distorted which fostered joint separation and prevented vertical and horizontal alignment, and finally, the ties were not "form fitted" to prevent slippage between the tie and face shell. Second, the ties were constructed of welded wire rods which are not conducive to a dry-stacked, groutless masonry system. Over time, the ties would fail through corrosion and the "Formwall" system would fail catastrophically.

To preclude these problems, it is recommended that the ties be fabricated using a noncorrosive material which can be pultruded or extruded through a die matching the geometric shape of the dovetail channels of the face shell. Moreover, the ties should be available in heights (lengths) which tie three subsequent courses together. In other words, the first tie should extend half a face shell (approximately 12") above the first course. The second tie should extend half a face shell above the third course (approximately 16") to tie together the second, third, and fourth courses. The 16" tie will be repeated until the final course which receives a 4" tie. Obviously, for this scheme to work, the actual height of the face shell must be 8". With the final course in place, the wall is capped with a solid concrete masonry unit or precast cap to maximize the distribution of the post-tensioning force.

6.3 Summary

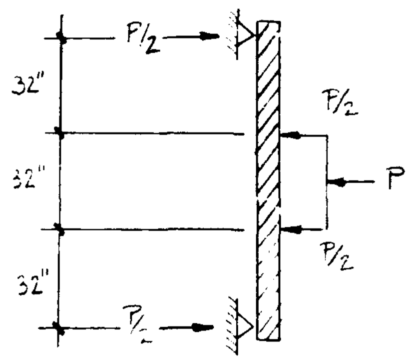
The "Formwall" panels as tested were not capable of developing resistance to flexural loads. However, with some physical modifications, theory predicts that they can sustain a lateral load of sufficient magnitude for practical use. The exact value depends upon future research in the field of dry-stacked masonry and the implementation of physical changes to the "Formwall" units similar to those noted in this report. When perfected, post-tensioned dry-stacked masonry could result in significant savings for roadside barriers, landscaping elements, basement walls for low income housing, and temporary structures.

Appendix A

PRELIMINARY CALCULATIONS

A.1 Nonreinforced Panel

Determine the maximum load for the nonreinforced panel shown below. The panel is 4′ x 8′ x 6″ thick, hollow load-bearing concrete masonry units, and Type "S" mortar. Weight of the CMU was approximated at 50 psf and the h/t ratio is 17. The allowable working stresses for compression and tension in flexure for Type "S" mortar are 150 psi and 24 psi, respectively. Properties for the wall panel are as indicated.



$A = 96.0\ in^2$

$S = 185.2\ in^3$

$M = Pa = 16P$

$f_t = 24.0$ psi

$w = 0.5\ (8)\ (50) = 200$ lb/ft

Including self weight of half the wall panel,

$$f_t = -w/A + M/S \tag{A.1}$$

where

f_t = allowable flexural tensile stress in masonry in in,

w = weight of wall in lb/ft,

A = effective area in in^2,

M = applied moment in in-lb

S = section modulus in in^3.

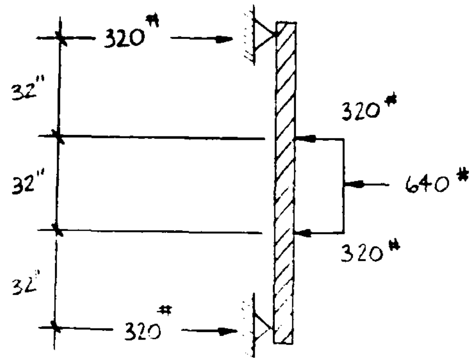
Hence,

$$24.0 = -200/96.0 + 16P/185.2$$

$$P = 301.91 \text{ lb}$$

A.2 "Formwall" Panel, Full Composite Action (Best Case Scenario)

Assuming full composite action between the ties and face shells, determine the post-tensioning force required to resist an applied lateral load of 20 psf. The lateral load is applied as two equal concentrated loads symmetrically placed. See diagram below. Properties for the "Formwall" units are as indicated (Appendix B).



A = 61.01 in^2

S = 116.12 in^3

M = 10,420 in-lb

The post-tensioning force can be determined from

$$f_t = F/A - M/S \qquad \text{(A.2)}$$

where

f_t = stresses in outer fibers on tensile face in psi,

F = post-tensioning force in lb,

A = effective area in in^2,

S = section modulus in in^3,

M = applied moment in in-lb.

Hence,

ft = 0 = F/61.01 - 10,420/116.12,

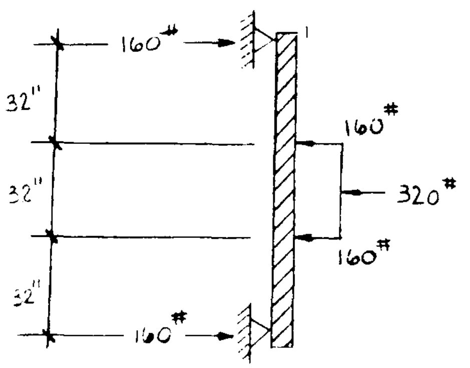
F = 5,380.15 lbs,

or

F = 2,690 lbs per tendon.

A.3 "Formwall" Panel, Noncomposite Action (Worst Case Scenario)

Assuming the same loading conditions, determine the post-tensioning force required if there was no composite action between the ties and face shells. Assume that the ties distribute the applied moment equally between the two face shells of the wall panel. Properties for the "Formwall" unit are as indicated.



A = 31.01 in2

S = 1.105 in3

M = 5,120 in-lb

The post-tensioning force can be determined from equation (A.2)

$$f_t = 0 = F/A - M/S$$

$$F = (5{,}120 \times 31.01) / 1.105$$

$$F = 143{,}684 \text{ lbs},$$

or

$$F = 71{,}842 \text{ lbs per tendon.}$$

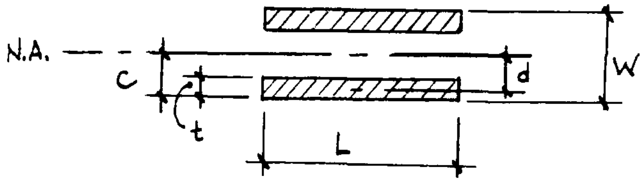
A.4 Recommended Post-Tensioning Force

Obviously, there is a significant difference between full composite action and the face shells acting independently. While full composite action was not assumed, a modest post-tensioning force of 3,500 pounds per tendon was targeted. This value was based upon 25 percent of the allowable axial compressive stress (F_a) and a margin of safety.

Appendix B

ASSUMED GEOMETRIC PROPERTIES OF THE "FORMWALL" UNITS

B.1 Best Case Scenario (Composite Action)



where

t = effective thickness in in,

d = distance from the neutral axis of the two face shells and the centroid of the effective bearing surface in in,

L = effective length in in,

W = effective width in in,

A = effective area ($t \times L \times 2$) per unit in in^2.

Thus,

$$S = I/c$$

where

S = section modulus in in^3,

I = moment of inertia ($\Sigma Io + \Sigma Ad^2$) in in^4,

c = distance from the neutral axis of the two face shells and the extreme fibers in in.

B.1.1 Example Problem

Determine the geometric properties for the "Formwall" unit using the average value of the three sample units from Table 3.2 of Section 3.2.1.2, Dimensional Evaluation of the "Formwall" Units. Thus,

$$t = 0.646''$$

$$L = 15.883''$$

Therefore,

$$A = 10.26 \text{ in}^2 \text{ per face shell,}$$

or

$$A = 20.52 \text{ in}^2 \text{ per "Formwall" unit.}$$

The moment of inertia then, is

$$I = I + Ad^2,$$

where

$$I = 2 \times (1/12 \times L \times t3) = 0.714 \text{ in}^4,$$

$$Ad2 = 2 \times (10.26 \times 2.153) = 95.12 \text{ in}^4,$$

hence

$$I = 95.83 \text{ in}^4 \text{ per "Formwall" unit.}$$

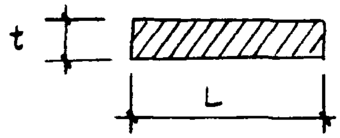
Consequently, the section modulus for the wall panel is

$$S = (95.83 \times 3 \text{ units per panel}) / 2.48,$$

or

$$S = 116.12 \text{ in}^3.$$

B.2 Worst Case Scenario (Noncomposite Action)



$$S = 1/6\, L\, t^2$$

$$S = 1.105 \text{ in}^3.$$

Appendix C

CERTIFICATES OF COMPLIANCE WITH ASTM STANDARDS

This appendix contains certificates and test reports. These documents demonstrate that the conventional concrete masonry units, mortar, and sand comply with the appropriate ASTM standard as specified in each document.

E. DeVECCHIS & SONS, INC.
PCMA
ESTABLISHED 1925
11 NORTH DEPOT STREET • MOUNT UNION, PENNSYLVANIA 17066

CONCRETE MASONRY UNITS CERTIFICATE

CONTRACTOR:

JOB NAME:

GENTLEMEN:

We hereby certify that all concrete masonry load-bearing units hollow & solid shall meet the following ASTM specificstions.

ASTM C90-70 Hollow load-bearing concrete masonry units

ASTM C145-71 Solid load-bearing concrete masonry units

ASTM C55-71 Concrete masonry bricks

The above units to meet the following classifications.

Units tested for compressive strength in accordance with ASTM method of test desingnation C140.

The above units to meet the following classification No 2.

2.1.1. Grade N for general use such as in exterior walls below & above grade that may or may not be exposed to moisture penetration or the weather & for interior wall & back-up.

2.1.2. Grade S limited to use above grade in exterior walls with weather-protective coating & in walls not exposed to the weather.

All units are cured 30 days prior to delivery to job site.

Sincerely,

E. DeVecchis & Sons, Inc.

Elegance in building materials for the residential, commercial, & industrial trades.

TELEPHONES: MOUNT UNION 814-542-[illegible] · LEWISTOWN 717-[illegible] · HOLLIDAYSBURG 814-[illegible] · STATE COLLEGE 814 [illegible]

McCREATH LABORATORIES, INC.
610 WILLOW STREET
HARRISBURG, PENNSYLVANIA 17101
PHONE [illegible]

REPORT OF TESTS

March 1, 1989

Lab No. 26136B

E. Devecchis & Sons, Inc.
P. O. Box 733
State College, Pennsylvania 16804-0733

Gentlemen:

On February 16, 1989 we received from your company three (3) (6" x 8" x 16" - 3 Core) Masonry Units to be tested for Compressive Strength in accordance with A.S.T.M. Method of Test Designation C140. The results are as follows:

Unit Ident.	Total Applied Load (Lbs.)	Gross Area (In.^{2})	Gross Compressive Strength (P.S.I.)	Net Compressive Strength (P.S.I.)
1B	292,000	87.9	3320	5710
2B	286,000	87.9	3250	5600
3B	304,500	87.9	3460	5960
Ave.			3340	5760

These Units have a Net Area of 58.1% (Of Gross)

Respectfully submitted,

McCREATH LABORATORIES, INC.

ALLENTOWN CEMENT CO., INC.

Mr. Wm. F. Lower
E. Devecchis & Sons
Box 733
State College, PA 16804

Dear Mr. Lower:

This is to certify that Allentown Types S and N masonry cements manufactured by Allentown Cement Co., Inc. at Evansville, PA complies with the requirements of A.S.T.M. C91 and Federal SS-C-1960/1 for masonry cements.

When mixed according to A.S.T.M. C270 with an approved masonry sand meeting specification C144, it will produce Types S and N type mortar.

Very truly yours,

Louis A. Jany

Louis A. Jany
Quality Control Manager

LAJ:dmh

cc: (8) Above
(1) BJI

Schadt Avenue Office Building • Schadt Avenue & Mickley Road • Whitehall, PA 18052 4587 • 215 264 4480, 800 322 9620

EASTERN INDUSTRIES, INC.
A DIVISION OF STABLER COMPANIES INC
P O BOX 177 WINFIELD PA 17889 0177
PHONE (717) 524 2251
FAX (717) 524 7995

June 14, 1991

E. Devecchis & Sons, Inc.
P. O. Box 733
State College, PA 16804

SUBJECT: A.S.T.M. C144 Sand Certifications

PROJECT: E. Devecchis & Sons, Inc.

Gentlemen:

We hereby certify that the sand to be supplied to the above mentioned project form our Strodes Mills Quarry meets the requirements of A.S.T.M C144. If you have any questions or comments, please forward to the Winfield office at P. O. Box 177, Winfield, PA 17889-0177. Our telephone number is (717) 524-2251.

Yours very truly,

EASTERN INDUSTRIES, INC.

GREGORY L. BROUSE
Materials Engineer

GLB/bg

SWORN TO AND SUBSCRIBED BEFORE ME THIS 14th DAY OF JUNE, 1991.

Cherry Fisher Berg

NOTARIAL SEAL
CHERRY FISHER BERG, Notary Public
Union Twp, Union County, Pa
My Commission Expires May 01, 1995

Appendix D

SECTION PROPERTIES OF CONVENTIONAL MASONRY UNITS

This appendix contains a copy of NCMA-TEK Bulletin 141A, "Concrete Masonry Section Properties for Design." This bulletin specifies the section properties used in the preliminary calculations for the conventional wall panel. This bulletin is part of an information series from the National Concrete Masonry Association.

NCMA-TEK

An Information series from National Concrete Masonry Association

CONCRETE MASONRY SECTION PROPERTIES FOR DESIGN

TEK 141A

Key Words: section modulus, moment of inertia, net area, flexural compressive stress, flexural tensile stress, face shell mortar bedding, full mortar bedding, fully grouted, partially grouted, radius of gyration, effective width of compression zone, face shell thickness, nominal width, specified width

INTRODUCTION

The information in this TEK was developed to be used as an aid by design professionals in the structural design of concrete masonry. Calculated section properties for single wythe concrete masonry walls are presented based upon minimum face shell and web thickness requirements of ASTM Specification C90[1]. Net cross sectional area, A_n, moment of inertia, I, section modulus, S, and radius of gyration, r, for five typical wall thicknesses are covered. Tables include section properties along the vertical and horizontal axes (Figure 1).

The method used to calculate stresses in accordance with these section properties is described in the following section. The basis upon which the section properties were determined is illustrated in Figure 2. Design stresses are required to be within the allowable stress limitations permitted by masonry building codes.[2,3,4,5]

CALCULATING DESIGN STRESSES

Axial Compressive Stress

Axial compressive stress is based on distributing the axial load (P) over the net cross sectional area (A_n) of the member.

$$f_a = P/A_n$$

The maximum value of f_a occurs at the minimum section of the member. For ungrouted or partially grouted walls constructed with face shell mortar bedding, the minimum section occurs through the mortar joint. The net cross sectional area, A_n, through the mortar joint is equal to the area of the two face shells plus the cross sectional area of the grouted cells and the area of the mortared webs immediately adjacent to the grouted cells. The value of A_n for solid walls, including solid grouted walls, is based on the specified width of the member times the length of the member.

Flexural Compressive Stress

Flexural compressive stress in uncracked masonry walls is based on the moment (M) divided by the section modulus (S) of the member.

$$f_b = M/S$$

The maximum flexural compressive stress occurs at the minimum section of the member described previously.

Combined Axial and Flexural Compressive Stress

The combined axial and flexural compressive stress in uncracked masonry walls is based on the summation of the axial compressive stress (f_a) acting simultaneously with the flexural compressive stress (f_b).

$$f_a + f_b = P/A_n + M/S$$

Flexural Tensile Stress

Flexural tensile stress in uncracked masonry walls occurs on the opposite face of the wall from the flexural compression face and is based on the moment (M) divided by the section modulus (S).

$$f_t = M/S$$

The maximum flexural tensile stress occurs at the minimum section of the member.

Combined Axial Compression and Flexural Tension

The combined axial compression and flexural tension in uncracked masonry walls is based on the flexural tensile stress (f_t) acting simultaneously with the axial compressive stress (f_a).

$$f_t - f_a = M/S - P/A_n$$

Reinforced concrete masonry walls are designed based on a cracked section in which the tensile strength of masonry is neglected. Cracked concrete masonry wall section properties are not covered in this TEK. The section properties in the tables are used in determining the cracking level strength of reinforced walls as well as deflections of reinforced walls with uncracked sections.

PROPERTIES FOR DESIGN OF CONCRETE MASONRY WALLS

Table 1 — Walls Spanning Vertically

4 Inch Single Wythe Walls

Units	Grouted Cells	Mortar Bedding	A_n in^2/ft	I_x in^4/ft	S_x in^3/ft	r in
Hollow	None	Face Shell	18.0	38.0	21.0	1.35
Hollow	None	Full	21.6	39.4	21.7	1.35
Solid	None	Full	43.5	47.6	26.3	1.05
Hollow	8" o. c.	Face Shell	43.5	47.6	26.3	1.05
Hollow	16" o. c.	Face Shell	31.0	42.9	23.7	1.09
Hollow	24" o. c.	Face Shell	26.7	41.3	22.8	1.16
Hollow	32" o. c.	Face Shell	24.5	40.5	22.3	1.19
Hollow	40" o. c.	Face Shell	23.2	40.0	22.1	1.22
Hollow	48" o. c.	Face Shell	22.3	39.7	21.9	1.23
Hollow	56" o. c.	Face Shell	21.7	39.4	21.8	1.24
Hollow	64" o. c.	Face Shell	21.3	39.3	21.7	1.25
Hollow	72" o. c.	Face Shell	20.9	39.1	21.6	[illegible]

6 Inch Single Wythe Walls

Units	Grouted Cells	Mortar Bedding	A_n in^2/ft	I_x in^4/ft	S_x in^3/ft	r in
Hollow	None	Face Shell	24.0	130.3	46.3	2.08
Hollow	None	Full	32.2	139.3	49.5	2.08
Solid	None	Full	67.5	178.0	63.3	1.62
Hollow	8" o. c.	Face Shell	67.5	178.0	63.3	1.62
Hollow	16" o. c.	Face Shell	46.6	155.1	55.1	1.64
Hollow	24" o. c.	Face Shell	39.1	146.8	52.2	1.74
Hollow	32" o. c.	Face Shell	35.3	142.7	50.7	1.80
Hollow	40" o. c.	Face Shell	33.0	140.2	49.9	1.84
Hollow	48" o. c.	Face Shell	31.5	138.6	49.3	1.86
Hollow	56" o. c.	Face Shell	30.5	137.4	48.9	1.88
Hollow	64" o. c.	Face Shell	29.6	136.5	48.5	1.90
Hollow	72" o. c.	Face Shell	29.0	135.8	48.3	1.91

8 Inch Single Wythe Walls

Units	Grouted Cells	Mortar Bedding	A_n in^2/ft	I_x in^4/ft	S_x in^3/ft	r in
Hollow	None	Face Shell	30.0	308.7	81.0	2.84
Hollow	None	Full	41.5	334.0	87.6	2.84
Solid	None	Full	91.5	443.3	116.3	2.20
Hollow	8" o. c.	Face Shell	91.5	443.3	116.3	2.20
Hollow	16" o. c.	Face Shell	62.0	378.6	99.3	2.20
Hollow	24" o. c.	Face Shell	51.3	355.3	93.2	2.34
Hollow	32" o. c.	Face Shell	46.0	343.7	90.1	2.42
Hollow	40" o. c.	Face Shell	42.8	336.7	88.3	2.47
Hollow	48" o. c.	Face Shell	40.7	332.0	87.1	2.51
Hollow	56" o. c.	Face Shell	39.1	328.7	86.2	2.54
Hollow	64" o. c.	Face Shell	38.0	326.2	85.6	2.56
Hollow	72" o. c.	Face Shell	37.1	324.3	85.0	2.57

Table 2 — Walls Spanning Horizontally

4 Inch Single Wythe Walls

Units	Bond Beam	Mortar Bedding	A_n in^2/ft	I_y in^4/ft	S_y in^3/ft
Hollow	None	Face Shell	18.0	38.0	21.0
Solid	None	Full	43.5	47.6	26.3
Hollow	8" o. c.	Face Shell	43.5	47.6	26.3
Hollow	16" o. c.	Face Shell	30.2	42.8	23.6
Hollow	24" o. c.	Face Shell	26.1	41.2	22.8
Hollow	32" o. c.	Face Shell	24.1	40.4	22.3
Hollow	40" o. c.	Face Shell	22.9	40.0	22.1
Hollow	48" o. c.	Face Shell	22.1	39.6	21.9
Hollow	56" o. c.	Face Shell	21.5	39.4	21.7
Hollow	64" o. c.	Face Shell	21.0	39.2	21.6
Hollow	72" o. c.	Face Shell	20.7	39.1	21.6

6 Inch Single Wythe Walls

Units	Bond Beam	Mortar Bedding	A_n in^2/ft	I_y in^4/ft	S_y in^3/ft
Hollow	None	Face Shell	24.0	130.3	46.3
Solid	None	Full	67.5	178.0	63.3
Hollow	8" o. c.	Face Shell	67.5	178.0	63.3
Hollow	16" o. c.	Face Shell	44.7	154.2	54.8
Hollow	24" o. c.	Face Shell	37.8	146.2	52.0
Hollow	32" o. c.	Face Shell	34.4	142.3	50.6
Hollow	40" o. c.	Face Shell	32.3	139.9	49.7
Hollow	48" o. c.	Face Shell	30.9	138.3	49.2
Hollow	56" o. c.	Face Shell	29.9	137.1	48.8
Hollow	64" o. c.	Face Shell	29.2	136.3	48.5
Hollow	72" o. c.	Face Shell	28.6	135.6	48.2

8 Inch Single Wythe Walls

Units	Bond Beam	Mortar Bedding	A_n in^2/ft	I_y in^4/ft	S_y in^3/ft
Hollow	None	Face Shell	30.0	308.7	81.0
Solid	None	Full	91.5	443.3	116.3
Hollow	8" o. c.	Face Shell	91.5	443.3	116.3
Hollow	16" o. c.	Face Shell	59.3	376.0	98.6
Hollow	24" o. c.	Face Shell	49.5	353.6	92.7
Hollow	32" o. c.	Face Shell	44.7	342.4	89.8
Hollow	40" o. c.	Face Shell	41.7	335.6	88.0
Hollow	48" o. c.	Face Shell	39.8	331.1	86.9
Hollow	56" o. c.	Face Shell	38.4	327.9	86.0
Hollow	64" o. c.	Face Shell	3[illegible].3	325.5	85.4
Hollow	72" o. c.	Face Shell	36.5	323.7	84.9

PROPERTIES FOR DESIGN OF CONCRETE MASONRY WALLS

Table 1 (Continued)
Walls Spanning Vertically

10 Inch Single Wythe Walls

Units	Grouted Cells	Mortar Bedding	A_n in^2/ft	I_x in^4/ft	S_x in^3/ft	r in
Hollow	None	Face Shell	33.0	566.7	117.8	3.55
Hollow	None	Full	50.4	635.3	132.0	3.55
Solid	None	Full	115.5	891.7	185.3	2.78
Hollow	8" o. c.	Face Shell	115.5	891.7	185.3	2.78
Hollow	16" o. c.	Face Shell	76.2	736.8	153.1	2.69
Hollow	24" o. c.	Face Shell	61.8	680.1	141.3	2.86
Hollow	32" o. c.	Face Shell	54.6	651.8	135.4	2.96
Hollow	40" o. c.	Face Shell	50.3	634.8	131.9	3.03
Hollow	48" o. c.	Face Shell	47.4	623.4	129.5	3.07
Hollow	56" o. c.	Face Shell	45.3	615.3	127.9	3.11
Hollow	64" o. c.	Face Shell	43.8	609.2	126.[illegible]	3.14
Hollow	72" o. c.	Face Shell	42.6	604.5	125.6	3.16

12 Inch Single Wythe Walls

Units	Grouted Cells	Mortar Bedding	A_n in^2/ft	I_x in^4/ft	S_x in^3/ft	r in
Hollow	None	Face Shell	36.0	929.4	159.9	4.29
Hollow	None	Full	57.8	1064.7	183.2	4.29
Solid	None	Full	139.5	1571.0	270.3	3.36
Hollow	8" o. c.	Face Shell	139.5	1571.0	270.3	3.36
Hollow	16" o. c.	Face Shell	90.2	1265.2	217.7	3.19
Hollow	24" o. c.	Face Shell	72.1	1153.3	198.4	3.39
Hollow	32" o. c.	Face Shell	63.1	1097.3	188.8	3.52
Hollow	40" o. c.	Face Shell	57.7	1063.7	183.0	3.60
Hollow	48" o. c.	Face Shell	54.1	1041.3	179.2	3.66
Hollow	56" o. c.	Face Shell	51.5	1025.3	176.4	3.70
Hollow	64" o. c.	Face Shell	49.5	1013.4	174.3	3.73
Hollow	72" o. c.	Face Shell	48.0	1004.0	172.7	3.76

Table 2 (Continued)
Walls Spanning Horizontally

10 Inch Single Wythe Walls

Units	Bond Beam	Mortar Bedding	A_n in^2/ft	I_y in^4/ft	S_y in^3/ft
Hollow	None	Face Shell	33.0	566.7	117.8
Solid	None	Full	115.5	891.7	185.3
Hollow	8" o. c.	Face Shell	115.5	891.7	185.3
Hollow	16" o. c.	Face Shell	72.3	729.2	151.5
Hollow	24" o. c.	Face Shell	59.2	675.0	140.3
Hollow	32" o. c.	Face Shell	52.7	648.0	134.6
Hollow	40" o. c.	Face Shell	48.7	631.7	131.3
Hollow	48" o. c.	Face Shell	46.1	620.9	129.0
Hollow	56" o. c.	Face Shell	44.2	613.1	127.4
Hollow	64" o. c.	Face Shell	42.8	607.3	126.2
Hollow	72" o. c.	Face Shell	41.7	602.8	125.3

12 Inch Single Wythe Walls

Units	Bond Beam	Mortar Bedding	A_n in^2/ft	I_y in^4/ft	S_y in^3/ft
Hollow	None	Face Shell	36.0	929.4	159.9
Solid	None	Full	139.5	1571.0	270.3
Hollow	8" o. c.	Face Shell	139.5	1571.0	270.3
Hollow	16" o. c.	Face Shell	85.3	1250.2	215.1
Hollow	24" o. c.	Face Shell	68.9	1143.3	196.7
Hollow	32" o. c.	Face Shell	60.7	1089.8	187.5
Hollow	40" o. c.	Face Shell	55.7	1057.7	182.0
Hollow	48" o. c.	Face Shell	52.4	1036.3	178.3
Hollow	56" o. c.	Face Snell	50.1	1021.1	175.7
Hollow	64" o. c.	Face Shell	48.3	1009.6	173.7
Hollow	72" o. c.	Face Shell	47.0	1000.7	172.2

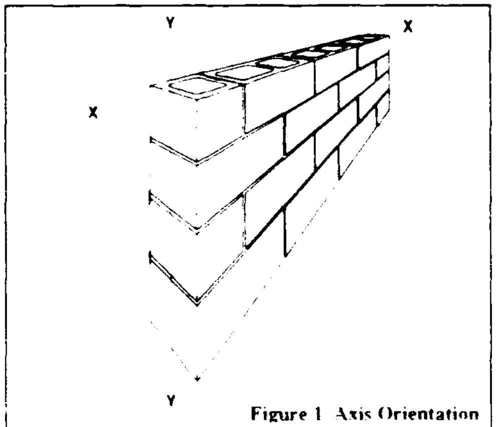


Figure 1 Axis Orientation

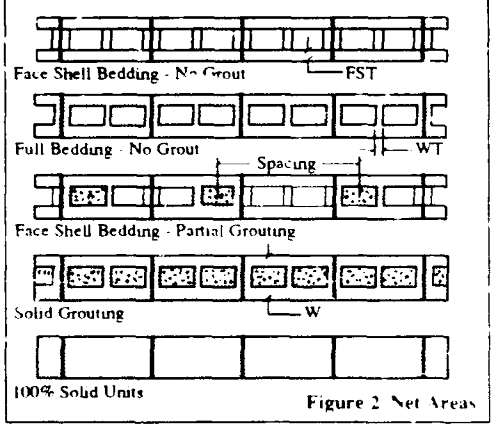


Figure 2 Net Areas

Shear Stress

Shear stress in uncracked walls is determined by the following:

$$f_v = VQ/It$$

The maximum shear stress calculated by this equation for shear walls without flanges is:

$$f_v = (3/2)\ V/A_n$$

Some building codes[3] incorporate the 3/2 coefficient into the allowable stress and thus determine shear calculations on the average shear stress, V/A_n. The average shear stress is also used to limit shear acting normal to the plane of the wall and for critical sections of cantilevered and corbelled sections.

Table 3 - Dimensions for Concrete Masonry Walls[1]

Nominal Width inches	Actual Width (W) inches	Face Shell Thickness (FST) inches	Web Thickness (WT) inches
4	3 5/8	3/4	3/4
6	5 5/8	1	1
8	7 5/8	1 1/4	1
10	9 5/8	1 3/8	1 1/8
12	11 5/8	1 1/2	1 1/8

(1) These dimensions are minimum dimensions permitted by ASTM C90. Properties are applicable for units complying with ASTM C90.

Radius of Gyration

Allowable compressive stress values are based on the slenderness of the member. Some codes[3] determine slenderness effects based on wall width, W; other codes [2] use radius of gyrations, r.

$$r = (I/A)^{1/2}$$

Radius of gyration, r, is based on the average cross section which is taken through the masonry unit. Therefore, the webs and face shells are included in the calculations of I_x and A_x when calculating r. Calculations are based on two-core units (units having three cross webs) with face shell and web thicknesses shown in Table 3.

Notations:

- A_n = net cross-sectional area of masonry, in.^{2}
- f_a = calculated compressive stress in masonry due to axial load only, psi
- f_b = calculated compressive stress in masonry due to flexure only, psi
- f_t = calculated tensile stress in masonry, psi
- f_v = calculated shear stress in masonry, psi
- I = moment of inertia of masonry, in.^{4}
- M = maximum moment occurring simultaneously with design shear force V at the section under consideration, in.-lb
- P = axial load, lb
- Q = first moment about the neutral axis of a section of that portion of the cross section lying between the plane under consideration and extreme fiber, in.^{3}
- r = radius of gyration, in.
- S = section modulus, in.^{3}
- t = effective thickness of section when shear is to be calculated, in.
- V = design shear force, lb

References:

(1) ASTM C90, Standard Specification for Hollow Loadbearing Concrete Masonry Units
(2) ACI 530-88/ASCE 5-88, Building Code Requirements for Masonry Structures
(3) International Conference of Building Officials (ICBO) Uniform Building Code, Chapter 24
(4) Building Officials and Code Administrators International Inc. (BOCA/National Building Code), Article 14
(5) *Southern Building Code Congress International (Standard Building Code), Article 14*

NATIONAL CONCRETE MASONRY ASSOCIATION
P.O. Box 781, Herndon, Virginia 22070

BIBLIOGRAPHY

Al-Manaseer, A. A. and V. V. Neis. "Load Tests on Post-Tensioned Masonry Wall Panels." ACI Structural Journal (November-December 1987): 467-472.

American Concrete Institute and American Society of Civil Engineers. 1990. Commentary on Building Code Requirements for Masonry Structures. (ACI 530-88/ASCE 5-88).

Horton, Richard T. and Maher K. Tadros. "Deflection of Reinforced Masonry Members." ACI Structural Journal (July-August 1990): 453-463.

Ganz, Hans-Rudolf. 1989. "New Post-Tensioning System For Masonry." Paper presented at the 5th Canadian Masonry Symposium, 5-7 June, at University of British Columbia, Vancouver, B.C., Canada.

Jones, L. L. 1962. Ultimate Load Analysis of Reinforced and Prestressed Concrete Structures. Frederick Ungar Publishing Co., New York.

Lenczner, David. 1972. Elements of Loadbearing Brickwork. Pergamon Press, New York.

Libby, James R. 1990. Modern Prestressed Concrete, Fourth Edition. Van Nostrand Reinhold, New York, New York.

Moore, J. F. A. "The resistance of masonry to lateral loading." The Structural Engineer (November 1986): 319-320.

NCMA-TEK Bulletin 7b. 1980. "Loadbearing Concrete Block in High Rise Buildings." National Concrete Masonry Association, Herndon, Virginia.

NCMA-TEK Bulletin 15. 1969. "Compressive Strength of Concrete Masonry." National Concrete Masonry Association, Herndon, Virginia.

NCMA-TEK Bulletin 20B. 1984 (Rev. 1989). "Mortars for Concrete Masonry." National Concrete Masonry Association, Herndon, Virginia.

NCMA-TEK Bulletin 22A. 1979. "Prism Testing for Engineered Concrete Masonry." National Concrete Masonry Association, Herndon, Virginia.

NCMA-TEK Bulletin 24. 1970. "Engineered Concrete Masonry--Wind Loads." National Concrete Masonry Association. Herndon, Virginia.

NCMA-TEK Bulletin 27. 1971. "Flexural Design of Nonreinforced Engineered Concrete Masonry." National Concrete Masonry Association, Herndon, Virginia.

NCMA-TEK Bulletin 34. 1971. "Combined Loads On Concrete Masonry Walls." National Concrete Masonry Association, Herndon, Virginia.

NCMA-TEK Bulletin 36A. 1986. "ASTM Specifications for Concrete Masonry Units." National Concrete Masonry Association, Herndon, Virginia.

NCMA-TEK Bulletin 60A. 1990. "Specifications for Masonry Structures (ACI 530.1-88/ASCE 6-88)." National Concrete Masonry Association, Herndon, Virginia.

NCMA-TE' Bulletin 63. 1975. "Partially Reinforced Concrete Masonry Walls. National Concrete Masonry Association, Herndon, Virginia.

NCMA-TEK Bulletin 70A. 1986. "Concrete Masonry Prism Strength." National Concrete Masonry Association, Herndon, Virginia.

NCMA-TEK Bulletin 73. 1975. "Empirical Design Of Concrete Masonry Walls." National Concrete Masonry Association, Herndon, Virginia.

NCMA-TEK Bulletin 84. 1977. "Strength and Time-Dependent Deformations of Reinforced Concrete Masonry." National Concrete Masonry Association. Herndon, Virginia.

NCMA-TEK Bulletin 108. 1979. "Testing Concrete Masonry Assemblages." National Concrete Masonry Association, Herndon, Virginia.

NCMA-TEK Bulletin 113A. 1977 (Rev 1990). "Building Code Requirements Herndon, Virginia.

NCMA-TEK Bulletin 130. 1983. "Concrete Masonry Unit--Testing Variables." National Concrete Masonry Association. Herndon, Virginia.

NCMA-TEK Bulletin 141A. 1984 (Rev. 1990). "Concrete Masonry Section Properties for Design." National Concrete Masonry Association, Herndon, Virginia.

NCMA-TEK Bulletin 145. 1985. "Glossary of Concrete Masonry Terms." National Concrete Masonry Association, Herndon, Virginia.

NCMA-TEK Bulletin 166. 1991. "Product Certification." National Concrete Masonry Association, Herndon, Virginia.

Orton, Andrew. 1986. Structural Design of Masonry. Longman Inc., New York.

Ostag, William P. 1986. Post-Tensioned Single Wythe Concrete Masonry Walls. Master's thesis, Department of Architectural Engineering, The Pennsylvania State University, University Park, Pennsylvania.

Page, A. W. and N. G. Shrive. "Concentrated Loads on Hollow Masonry--Load Dispersion Through Bond Beams." The Masonry Society Journal (July-December 1987): T45-T51.

Post-Tensioning Institute. 1976. Post-Tension Manual. Glenview, Illinois.

Preston, H. Kent and Norman J. Sollenberger. 1967. Modern Prestressed Concrete. McGraw-Hill Book Company, New York, New York.

Schneider, Robert A and Walter L. Dickey. 1987. Reinforced Masonry Design. Second Edition. Prentice-Hall, Inc., Englewood Cliffs, New Jersey.

Simbeya, K.W., A.E. McMullen, E.L. Jessup and N.G. Shrive. "Stress Distributuion in Hollow Masonry Due to Concentrated Loading." The Masonry Society Journal (July-December 1987): T25-T38.

VSL International LTD. "Post-Tensioned Masonry Structures." VSL Report Series. Berne, Switzerland.